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FEDERAL RESPONSIBILITY FOR FLOOD CONTROL

By the Hon. James J. Davis[1]

Since Colonial times, floods have taken their toll annually in various sections of the country. Experience in Pennsylvania is typical, and a few comments on conditions in that State will suffice to indicate the importance of the problem, not only there, but throughout the United States.

Flood Problems in Pennsylvania

Prior to 1937, about thirty major floods had been recorded on the Delaware River, thirty-five on the Susquehanna, and about fifty on the Ohio. (These three streams together drain approximately 95% of the State.) Numerous floods localized on their smaller tributaries have also occurred frequently and have caused serious damage.

The flood of June 1, 1889, is the greatest on record in the Susquehanna Basin. Although the storm was concentrated over the water-sheds of the Juniata and the West Branch of the Susquehanna River, most of the central and eastern part of the State suffered unparalleled losses; at Johnstown, in Cambria County, 2 142 lives were lost, and a large part of the city was destroyed.

The Pittsburgh Flood Commission estimated the flood toll to the City of Pittsburgh alone, from 1898 to 1908, at $12 000 000, of which $6 500 000 was caused by three floods in 1907 and 1908. The total financial loss along the Allegheny River resulting from the 1913 flood amounted to $720 000. Damage in the Shenango and Beaver Valleys from the same flood was $2 100 000. In August, 1915, in the City of Erie, thirty-four people were drowned and damage amounting to $2 000 000 resulted from a flood in Mill Creek. In July, 1931, Norristown suffered a loss of $1 000 000 as the result of floods in two small creeks traversing the Borough. York lost $260 000 in 1884 and $4 360 000 in 1933 from floods in Codorous Creek.

Even these figures, of course, become insignificant in comparison with those for the general floods of 1936, in which Pennsylvania was one of the heaviest sufferers. The flood damages throughout the East during that year are variously estimated at from $250 000 000 to $500 000 000.

Floods can be controlled by storage or retarding basins, diversion channels, stream channel improvements, and levees or embankments. Protection generally may best be secured, not by any one of these methods, but by a combination of several of them. For example, storage basins, together with certain channel improvements, have been recommended for flood protection for the City of Pittsburgh. A combination of channel improvement and levee or dike construction has been suggested along the North Branch of the Susquehanna River to protect the Wyoming Valley. A combination of retarding basins, diversions, and channel improvements has been in successful operation for years at Harrisburg, Pa.

[1] U. S. Senator from Pennsylvania, Pittsburgh, Pa.

In the Shenango and Beaver Valleys, in Western Pennsylvania, flood control by storage basin retardation alone has been effected by the completion of the State Pymatuning Reservoir Project, although the primary purpose of this reservoir is to provide sufficient water during the dry season for domestic and industrial use.

A carefully determined public policy for control and supervision of the streams of the Commonwealth was adopted and put into effect in 1933, with the enactment of the Dam and Encroachment Act, which made it unlawful for any one to construct any "water obstruction" without a State permit. Since that time, stream channels of a section adequate to discharge the maximum expected floods have been conserved and protected. Channels that had been encroached upon and restricted so that they were unable to pass floods without overflowing, have not been allowed to become worse, and, in various places, serious obstructions to the flow have been modified or removed and the channels have been improved. In communities subject to flood damage the State has made a number of surveys and hydraulic studies. Channel lines, marking the permissible limits of new construction, have been established in Johnstown, York, Manayunk, Sharon, New Castle, Butler, Reading, Norristown, Scranton, and Erie, and existing encroachments on these lines are gradually being removed.

The major flood-control problem in Pennsylvania—protection for Pittsburgh and the Upper Ohio River—was studied under the direction of the Pittsburgh Flood Commission from 1908 to 1912. In the latter year, the Commission recommended the construction of seventeen retarding or flood-storage reservoirs distributed over the drainage areas of the Allegheny and Monongahela Rivers, supplemented by a river wall at Pittsburgh. The cost was estimated to be about one-half the direct loss that would otherwise be caused by flood damage to the city within a 20-yr period. Between 1924 and 1929, United States Army Engineers studied the problem and developed many additional valuable data.

State legislation approved in 1931 authorized the Water and Power Resources Board to take up the study and develop a plan of flood control, and empowered it to proceed on its own authority, or at the request of some outside person or agency, to improve stream channels, build levees and diversion channels, and to construct retarding storage basins for flood control. The Board and its agencies were further empowered to "enter upon, take, appropriate or injure any land or lands", and damages sustained thereby are to be paid by the Department of Forests and Waters. In 1933, the Commission submitted a new plan, based upon all the essential data developed to date, providing for the construction of ten storage reservoirs, seven in the Allegheny water-shed and three in the Monongahela water-shed, at an estimated cost of $57 500 000.

It is natural to ask why a project so meritorious as this one has remained so many years in the planning stage. The answer is plain. The Allegheny River has portions of its water-shed in two States—New York and Pennsylvania. The Monongahela River has its water-shed in three States—Mary-

land, West Virginia, and Pennsylvania. The proposed reservoirs would be in several States and would affect the flow of the stream in each. It is not possible, therefore, for any city or county, or even for the State of Pennsylvania, to undertake to complete the project in its entirety, or in the comprehensive manner necessary, without assuming more responsibility than it rightly should. National control and action are necessary. Prior to 1936, however, the interest of the Federal Government in large streams was confined, except in a few cases, to the problems of navigation. The main question of concern to the National Government was how much the destruction of a forest, or the building of a bridge or a reservoir, would affect navigation.

More recently, however, a new consciousness has been developing. It is now recognized that flood control is a problem of major national significance, and that it deserves consideration in its own name and on its own merit. It is recognized that what is done to control floods in one part of the country vitally affects the lives, the property, and the well-being of people in other parts.

Federal Participation in Flood Control

The first aid from the Federal Government relating to flood control pertained to the Mississippi River and was provided in the Swamp Land Acts of 1849 and 1850. By these Acts Congress granted to the several States the swamp and overflowed lands within their borders and provided that the proceeds from the sale of the lands must be spent for drainage and flood protection. Congress passed another Act in 1850 that must be given an important place in any consideration of the problems of the Lower Mississippi Valley. This act "directed a topographical and hydrographical survey of the delta of the Mississippi River, with such investigation as might lead to determine the most practical plan for securing it from inundation." The report of Charles Ellet, a Civil Engineer working for the U. S. Army Engineers, was the result of this first official study of flood control by the Federal Government. The Ellet report came in 1852 and concluded that the control of the floods on the Mississippi was the nation's duty and that it was a question that "must be decided by the justice and humanity of the nation." That statement showed a definite trend in official thinking.

The Civil War halted the movement for flood control. In 1878, however, the Rivers and Harbors Bill provided $1 000 000 for aiding navigation on the Mississippi. The money was spent by the Board on the Improvement of the Mississippi River under the supervision of the U. S. Army Engineers. This Board was composed of Army engineers who favored levees for improving navigation. Thus, the Army engineers came to support those who wanted levees to control floods. The combining of the groups desiring flood control and the groups interested in navigation was evident in the debate on the bill creating the Mississippi River Commission in 1879, which put the United States definitely into flood-control work, and which probably stands as the most important piece of flood-control legislation in the entire history of the country.

One of the strongest arguments generally advanced for flood control by the Federal Government is that the United States owns the rivers and has paramount jurisdiction over them. The conclusion that the United States should not permit its property to damage the citizens of any section is the natural outgrowth of ownership. The interstate nature of the flood-control problem has furnished another reason for Federal responsibility. The contrast in size between the gigantic drainage area of the Mississippi River as a whole and the small territory drained by the Lower Valley—the region that suffers most from floods—is striking. The Mississippi Basin contains 1 250 000 sq miles of territory, 41% of the area of the United States. It includes all or parts of thirty-one States, the combined population of which is one-half that of the entire nation. The river system includes 15 000 miles of navigable streams and many thousands of miles of non-navigable ones. The contention that the interstate nature of the problem makes it a national one grows much stronger when it is understood that the area that suffers most from floods contributes little or none of the water that causes them.

The suggestion has frequently been made that the cost of flood control should be borne by the various States in proportion to their responsibility for the floods, but the difficulties of apportioning costs on such a basis appear to be insurmountable The State of North Dakota and the State of Pennsylvania, for example, both contribute water to the Mississippi, but it would hardly be possible to say how much the flow from either State adds to the floods on that stream. The problem is sufficiently complex when only the natural causes of floods are considered; it becomes much more so when the acts of States and individuals in regard to drainage and flood control are taken into account. Works built for flood protection by citizens of one State may pile up the water and spread it over the lands of the citizens of another State. On the other hand, the vigilant have often been inundated because of the negligence of their neighbors. Hence, the demand for Federal control.

Most of the people believe that the Federal Government exists for the practical purposes of protecting the general welfare. In considering the general welfare as related to flood control the humanitarian and the economic phases can not be entirely separated. The great sacrifices, suffering, expenses, and property losses of those at the mercy of the floods are all intertwined.

A consideration of Federal responsibility for flood control calls for thought as to the causes of floods. Apparently, all agree that the causes have many far-flung ramifications. They extend even to the grazing of cattle and sheep far in the interior, when over-grazing of the lands causes rapid run-off and erosion. Even the direction of a furrow in plowing may affect the quantity of water that flows from the surface of a field and the amount of sediment it carries. Paved streets and sewage and drainage systems have added at least to local flood problems by causing both a heavy and a rapid run-off. Many cities with splendid drainage systems do not realize that they have been hastening floods down upon their neighbors. Even the construction of hard-surfaced and well-drained highways throughout the country has

added to the areas from which run-off is heavy and rapid. Many of these highways have been built under Federal specifications and with the aid of Federal funds. Highways, railroads, cities, and industrial and commercial enterprises have aided in piling up the flood-waters by encroaching upon the natural channels of streams with bridges, embankments, piers, terminals, and other structures. In the case of navigable streams such encroachments have been made with the express permission of the Federal Government, which thereby has become an agent in creating these encroachments, and, therefore, in causing destructive floods.

However, the influence on floods of such factors as those just mentioned must be relatively small in comparison with that of the general deforestation, drainage, and development of the vast valley territory. Large areas of swamps and lowlands have been drained, forests have been cleared away, and the land has been used in such a way as to cause much erosion and rapid run-off. The hills pour torrents of turbid waters through man-made gullies into the rivers, filling their beds with mud, and causing them to overflow their banks.

Arguments for Federal Responsibility

Many believe that flood control is the duty of the Federal Government for the reason that no other power exists that is able to cope with the problem. In 1879, the Hon. James A. Garfield stated in the House of Representatives that it was "too vast for any State to handle; too much for any authority less than that of the nation itself to manage." When it is realized that the Federal Government, the States, levee boards, cities, counties, railways, and individuals have been building levees, and that ten agencies in the Departments of War, Interior, Commerce, and Agriculture, to say nothing of the PWA and WPA, have had authority in the development and control of streams, it appears rather remarkable that the task has been performed as well as it has; but a satisfactory public policy can never be developed under these conditions. Each organization naturally outlines its own problems and proceeds to work them out with little reference to those of other groups. This situation has caused a widespread demand for a unified control.

Added to these reasons for Federal responsibility in flood control are important considerations of interstate commerce, including the postal service, public health, and national defense. They are so obvious as to require no elucidation.

Federal control has been delayed because the construction of protective works costs large sums of money, because opponents have claimed that it constituted a reclamation project for the benefit of private property, because sectionalism has invested proposed flood-control legislation with the objectionable "pork-barrel" feature, because there has been disagreement as to the best methods of handling the problem, and because some have held that it was unconstitutional. None of these arguments has weighed so mightily as the general inertia of the public in forcing the issue. Organized representation of the flood-control cause in Washington, D. C., has not been sufficiently persistent or powerful.

Several strong points for the constitutionality of flood control by the Federal Government have been advanced, but the bulwark of most of the arguments has been the jurisdiction of the Federal Government over interstate commerce. The early participation of the United States in flood control was based entirely on the proposition of improving navigation to facilitate commerce on the Mississippi. Congress and the public paid particular attention to the provisions in the early appropriation acts that no part of the money should be spent for levees to prevent injury to lands by overflow. This point of view has long prevailed. However, in 1917, proponents of flood control sought to establish the right to control the Mississippi under the commerce clause on a wider basis than that of merely improving navigation. They frankly stated that it was a flood-control measure. They took the position that legislation for the improvement of navigation had been "predicated upon the power of Congress to regulate commerce", that the word, "navigation", did not appear in the Constitution but had been written in as a part of the interpretation of the commerce clause. They turned their attention to statements of legal authorities and to Court decisions, apparently with satisfactory results.

The general welfare clause has naturally furnished a strong point for those who sought to prove the constitutionality of flood control by the United States. It has been suggested that the Constitution has been expanded by usage and by interpretation to include many things under this head. To many the Government had as much right to make land suitable for habitation by protecting it against floods as it did to give away the public domain. Congress has been very liberal in voting money to relieve suffering among the victims of floods; little question as to the constitutionality of such aid has ever been raised. It has been urged with apparent logic that appropriations for flood protection mean as much for the general welfare as appropriations for the relief of flood sufferers and that they are, therefore, just as constitutional. Chief Justice Story maintained that if the benefit was general, whether it was located in "one State or several", Congress could appropriate money for it because it was for the general welfare.

The problem of flood control must be regarded fundamentally as one of engineering, and the Engineering Profession deserves recognition for the support it has given to the Federal program. Engineers have appeared times without number as witnesses before Congressional committees. They have passed many resolutions in State and National conventions demanding Federal control. No other group of financially disinterested persons has more generally urged complete control of the floods of the Mississippi and its tributaries.

The people of the United States have finally realized that floods will continue to increase in importance. Floods have not become more frequent, but as the population grows, they are becoming increasingly more destructive. Property values and the density of population have already reached such a status that millions of persons now live in areas that are without adequate protection. Flood control is now a national problem and must be considered fully as a Federal responsibility.

PROBLEMS IN DEVELOPING A NATIONAL FLOOD-PROTECTION POLICY

By Abel Wolman[2], M. Am. Soc. C. E.

Synopsis

After every major catastrophe public demand for preventive action reaches its peak and the solutions proposed are as varied as the interests involved. In this respect the March, 1936, floods were no exception. Demands for immediate action, for Congressional appropriations, for financial programs, filled the lay and technical journals. History teaches that in such periods of stress, judgment and logic are unduly influenced by emotion. It is frequently the most unfavorable time in which to formulate policies for control. When public interest is at its height, on the other hand, is the most fruitful period in which to make real progress in the solution of a difficult national problem.

Most thoughtful students of flood-protection procedure would probably agree on the following facts:

(*a*) Engineering studies of past destructive floods are by and large incomplete.

(*b*) Detailed and carefully prepared programs for immediate flood protection are likewise lacking in many areas, although reconnaissance studies are available.

(*c*) In the development of principles and policies much more than engineering information and conclusions is needed.

(*d*) A kind of statesmanship and creative thinking, not yet fully developed, is required, which will balance local and national needs and costs against assessments of benefits.

(*e*) A new sense of social responsibility on the part of local areas is needed.

(*f*) Protection for a local area should be designed not only to meet the flood-control requirements of the area, but for the social and economic advantage of the nation in respect to all water uses.

A large number of reports by Federal or local agencies have been prepared from time to time. The most important undertakings of this type are those known as the "308 Reports" of the Corps of Engineers, U. S. Army. Because of the limitations under which these documents were prepared, they cannot all be considered as of completely definitive character. In such instances adjustment is needed, and, in general, a continuing review is desirable.

The interval between the preparation of reports, where they have been made, and the actual initiation of construction work, is likely to be long.

[2] Chairman, Water Resources Committee, National Resources Committee, Washington, D. C.; Chf. Engr., State Dept. of Health, Baltimore, Md.

Almost a quarter of a century elapsed, for example, between the preparation of a document on the Muskingum River floods and the building of the control structures.

An analysis of national shortcomings in flood-protection measures soon discloses that the problem is highly complex and that it cannot be reviewed upon so simple a basis as flood protection *per se.* Although many engineers and most laymen would urge strongly the correction or prevention of flood damages, even a superficial review of the situation makes it clear that few flood-protection programs should proceed without consideration of the relationship of these measures to other possible uses of water. When such reviews are undertaken, simplicity of approach gives way to complexity of use and to conflicts in purpose and in policy.

Because of these difficulties in orientation the writer considers it wise to present a brief analysis of the various aspects of determining upon a national flood-protection policy and to outline what measures are under way, as of 1937, to clarify some of the problems involved. In this attempt, it will be apparent that considerations other than engineering loom large in the final solution. Nevertheless, any generalized program for water use must rest obviously upon detailed and careful engineering analysis. In periods of stress and haste this axiom has not always been observed. It is a prerequisite to all that is written in this paper.

Engineering Aspects

What are the links in the chain of engineering evidence that should be available before a national policy can emerge?

To engineers, of course, the first element of deficiency is in incomplete hydrologic data, not only for floods but for every form of river run-off. The Water Resources Committee of the National Resources Committee, in several of its sub-committee publications, has called attention to these deficiencies in basic hydrologic data. Whether these deficiencies are corrected in the future depends upon such organizations as the Society and upon lay support.

Without such book-keeping of hydrologic data, no policy can be adopted with complete confidence. The program should be nation-wide, continuous, and technically accurate, and the results should be regularly interpreted and reported to the profession. Discontinuous observation, although perhaps better than nothing, does not fully meet the requirements.

Even in 1937, detailed analyses of major floods leave something to be desired, although the "308" reports of the War Department and the publications by the Miami Conservancy District, the United States Geological Survey, and the Bureau of Agricultural Engineering, have added material of incalculable value in recent years. The studies should be extended. Findings are spotty geographically, and methods of analysis and of forecast still need improvement.

In determining the effects of different procedures on the control of floods, in some instances the profession is still divided, and the heat of discussion has so far prevented a calm analysis.

What part detention reservoirs, channel improvements, zoning control of encroachment, forestation, soil conservation, small dams, and so-called "up-stream engineering" should play in a general water-resources program, still remains to be demonstrated conclusively. The hiatus between claim and substantiation in fact is wide, and competitive claims are scarcely sufficient basis for a permanent policy. Only detailed field and office analysis of water resources control can disclose the relative merits of the various devices and procedures. In this regard, "newspaper experts" have not been the only delinquents. Members of the Engineering Profession have permitted themselves to be diverted into defensive or offensive controversial fields where their capacities for objective analysis have suffered or disappeared. At times, little or no interest has been exhibited by those individuals who should be most interested, in the multiple-use problem in water-resources development. Not all the present difficulties of approach may be ascribed to misguided enthusiasts among the laymen. After all, engineering guidance for public opinion is more necessary to-day than at any other time in the past. Aloofness, professional provincialism, political partisanship, are all strange bedfellows for engineering judgment. Engineering texts, of course, are filled with principles of caution and logic in the matter of water-resources development; but how frequently are they ignored! For example, the zoning principle suggested by the late Allen Hazen, M. Am. Soc. C. E., many years ago, for application in flood areas, when discussed at all to-day is met with scoffing in some groups and with complete disregard in others. Yet as a flood-protection measure, it may some day have the place in the sun that it undoubtedly deserves.

Financial Aspects

The engineering aspects of water-resources control lend themselves to quantitative determination and decision. This is not the case, however, with the financial aspect of the problem. The simple statement that water-resources development, and flood protection as a unit thereof, should be financed largely by local interests, is sound and clear on paper. No one can take issue with this statement of policy; but its practical application requires discussion.

It is not invariably true that a water-resources program is restricted to the confines of a single State. Often two or more States are involved. Often reservoir sites in one State may be used for the control of flow in three or four down-stream States. In this setting, the concepts of damages, benefits, and allocation of the costs of correction are decidedly confusing.

Any one reviewing the damages presumably caused by destructive floods is at once impressed with the absence of any standard criteria for evaluation. Any two agencies, Federal or local, are likely to arrive at separate estimates of the flood damages in a given area that will differ widely. These deficiencies in approach led the Water Resources Committee to appoint a sub-committee in December, 1936, to study the criteria for the evaluation of flood damages. No

one believes that an immediate standardized methodology in this field is desirable, but it should be possible to present certain major criteria of approach that would put such estimates on a more scientific basis. Likewise, no one believes that a methodology must be devised which is a substitute for long experience. An analysis of practical criteria may clarify, however, the concepts even for the experienced observer.

Most people assume that the economic justification for flood protection is simple to evaluate. It is surprising to find how little information is available, not only on specific flood damage, but on the general, special, and intangible benefits that might arise from the initiation of flood-protection measures. Various procedures have been used from place to place, but it is rare that the details of such principles of benefit evaluation are presented so as to be readily subject to check.

Since assessment of local benefit, presumably the base of local participation, is still in a nebulous state, the difficulty of adhering to what appears to be a sound policy becomes more complicated as each flood-protection project comes under review.

The pressing question of how much central responsibility for flood protection the Federal Government should assume, is more difficult to answer when viewed in the light of specific undertakings than when stated as a general principle. One reason for this difficulty is that the mere definition of "Federal" interest is continually undergoing adjustment. This is not solely due, as many believe, to the so-called rise in central bureaucracy. It may be the result and the penalty of increasing national integration. It may be the result and the penalty of growth of population and of the difficulty of separating local from regional hazards and corrective measures.

It is a Jeffersonian principle that government in the United States should result "in the partition of cares, descending in gradation from general to particular, that the mass of human affairs may be best managed for the good and prosperity of all." This precept is valid only until its application under the pressure of such difficulties as those encountered in water-resources development make deviations therefrom necessary. The question is how far such deviations are wise; how much may be accomplished within the framework of State autonomy; how well drainage-basin enterprises may be developed within that framework; and what re-adjustments in theoretical principles, if any, the pressing problems of water resources may make necessary.

In such a complicated problem it is obvious, at least to the writer, that slow motion is a sound principle of action. Heroic changes in procedures of Federal-local responsibility must be approached with caution. At the same time, it may be questioned whether such approaches should be so congealed by precedent no longer applicable, that no comprehensive treatment of water resources is possible.

In the allocation of financial responsibility, it should also be remembered that a simple formula may be a first necessity. The Flood Control Act of 1936 illustrates, however, that in the desire to adopt a simple formula for local participation, many inequities necessarily arise. The percentage to be

paid by local areas in terms of land and rights of way may vary from 10 to the maximum legal limit of 50. The intent of such a Congressional Act may be wise, but its application is likely to be inequitable. A logical substitute for the simple formula type of Congressional approach lies in moving administrative control. This gives rise in turn to the charge of increased bureaucracy, although the nature of the water-resources problem would appear to pre-suppose the necessity of avoiding a rigid formula for evaluating local and central responsibility.

When the problem is extended to the evaluation of benefits and costs in multiple uses of a stream, the difficulty of using a formula on a national basis becomes even more apparent. In this field, there may be no escape from continuing administrative adjustment and evaluation. This will no doubt create a variety of geographic apportionments of costs, and necessarily much argument. Here, again, is a fruitful field for the best technical thought of the country.

The formulation of a national fiscal policy with reference to any important public undertaking has always been beset with the necessity of reconciling logical processes with public sentiment and clamor. Changes, supported by local greed, may soon convert a sane national policy into a general "grab-bag" for distributing Federal funds to local projects, sometimes well conceived and involving important national interests, but too frequently badly conceived and purely local in character.

The cold statement of historical facts in regard to national enterprise may be interpreted as disheartening examples of "increasing feeding of local areas out of the Federal trough", or as examples of "the slow evolutionary assumption of Federal financial responsibility for undertakings of national interest." It is the writer's opinion that the correct interpretation lies somewhere between these two extremes.

Experience certainly teaches that before establishing any permanent national policy for financing public works, one should make certain, on the one hand, that the interests of the country as a whole are being adequately protected and advanced, and, on the other, that opportunities are not created for the complete degradation of local responsibility. The tendency, unfortunately, is generally in the latter direction, and safeguards must be provided in the water-resources development policy to avoid the error of expanding Federal power and financial responsibility without local responsibility.

It should be clear from this brief discussion of some of the financial problems involved that a trial-and-error technique is a pre-requisite to ultimate solution. Experimentation in government to-day arouses violent opposition in some quarters, but, carefully organized and carefully controlled, it appears to be a necessary adjunct to the normal processes of desirable water-resources development. A substitute for experimentation is no more desirable in this, than in any other scientific, field where rationalization has long since given way to trial-and-error technique.

More than fifty years ago, Ambassador Bryce made the discerning observation with reference to American political institutions, strongly applicable to

the subject in hand, that: "The longer one studies a vast subject, the more cautious in inference does he become."

Administrative Aspects

If future water development should result only from comprehensive study of possible multiple uses or methods, other problems beyond those of engineering and finance present themselves. Decisions must be reached from time to time as to what governmental or private agency should be permitted to undertake the construction of vast water-resources programs, whether for flood protection alone, or for multiple purposes. Controversy again rages in this field as between central and local public bodies and as between public and private agencies.

There are occasions, of course, when this particular administrative difficulty is absent. The development of flood-protection measures within Los Angeles County, California, alone, for example, may present no particular problems of national policy. The type of organization may be controlled locally. The methods of raising funds and assessments may be controversial, but, at the same time, not of national import. When the problem is extended, however, beyond the confines of a single county, or of a single State, and when the best uses of a stream include its development for power, for navigation, for potable water supply, and for flood protection, can some form of national control be avoided? The question is posed without prejudice at the moment. It should be brought clearly into the daylight, and its discussion should not rest wholly upon abstract and emotional concepts of political philosophy.

The administrative difficulties are not all resolved when the responsibility for the initial construction has been agreed upon. Similar difficulties arise in the operation and management of undertakings, where further reconciliation of diverging interests may be continually necessary.

Recent experience with one of the reservoirs in the Far West, is illustrative of a type of problem likely to be multiplied in the future. With a full reservoir, technical experts recommended, on the basis of snow surveys, that the reservoir be emptied so that its capacity would be available for catching spring run-off. If this had not been done the heavy spring run-off, equivalent to the reservoir capacity, would have created damaging floods. Yet compliance with this competent technical advice required extensive negotiations with political parties, private interests, and public agencies, leading ultimately to the Governor of the State for final decision. The engineer cannot ignore the brute facts of public pressure no matter how cloistered he may be.

History has shown that each drop of water may be the source of deadly conflict. The problems cannot be settled in the laboratory, but must be adjusted in terms of public demand. Undoubtedly, the Engineering Profession has the task of educating the public in the field of its operations. It is a task, incidentally, in which so far it has made no great progress; otherwise, the clamor for faulty engineering panaceas would not have reached the heights it has attained in recent years.

Suggested Approaches to the Development of a National Water Resources Policy

No easy road to the promulgation of a national policy for flood protection or for a national policy of broader character is available. Acceptance of this conclusion has led agencies of all levels of Government, and the Water Resources Committee and its predecessors (the Mississippi Valley Committee and the Water Planning Committee of the National Resources Board), to approach the task experimentally.

The operations of the Miami Conservancy District, the Muskingum Conservancy District, the Corps of Engineers on the Mississippi Valley flood-control work, and of the Tennessee Valley Authority are all indicative of a distinct, although unconscious, desire to direct public action into logical engineering, financial, and administrative channels. The varied experiences of each of these organizations disclose a gradual evolution toward emphasis on multiple uses of streams, on drainage-basin concepts, on Federal-local participations, and on generally broader handling of stream problems than those current in the early years of the century.

Whether for better or worse, people are beginning to view the control of a stream for flood reduction as an integral phase of the use of that stream for power, navigation, irrigation, public water supply, and sanitation. To the writer it is an evolutionary step of merit. It creates complications in all fields of attack, but the objectives appear wise.

To the aforementioned approaches by Federal and local agencies, the Water Resources Committee has added several undertakings, which are briefly presented herein as further examples of multiple-use of streams. In its studies, the Water Resources Committee has kept two basic principles in mind: First, it has included in the deliberations on the uses of a particular stream, the States covered by the drainage basin in question (this was done on the assumption that no real progress in analysis or solution of the problem would be successful without the highest degree of local participation); and, second, it has assumed that no standard method of approach has yet been evolved, which is invariably applicable to the United States as a whole.

The Committee assumed further that different problems would require different structural arrangements for experimental demonstration, and that the cost of carrying out such experimental studies in every instance should be borne by the Federal and local agencies primarily concerned.

The Red River of the North.—The investigation of the Red River of the North resulted largely from a meeting called on June 3, 1935, in Fargo, N. Dak., by the Governors of Minnesota and North Dakota to consider the problems of water conservation, stream pollution, and sewage disposal in the Red River Valley. The call was directed particularly to the members of the State Planning Boards of the three States—Minnesota, North Dakota, and South Dakota.

This conference was followed by a second one in St. Paul, Minn., on November 26, 1935, in which representatives of the National Resources Committee participated. At this meeting, a committee was appointed to develop

a co-ordinated plan for the development and protection of water throughout the basin of the Red River of the North. As representatives of the Water Resources Committee, W. W. Horner, M. Am. Soc. C. E., of St. Louis, Mo., and Harlan H. Barrows, of Chicago, Ill., were delegated to act. At the request of the conference, Mr. Horner was designated by the National Resources Committee as Technical Consultant in the preparation of the ultimate water plan.

On July 8, 1936, a report of the Third Inter-State Conference on the Red River of the North Drainage Basin was issued, proposing an adequately developed water plan, on a long-time basis, the primary objective of which is the development of dependable stream flows during the dry periods of the year in quantity ample for urban water supply and for the dilution of wastes. To attain this objective, the plan called for the development of storage in head-water areas, the regulation of stream flow in accordance with a predetermined program, and for improvement of stream channels to minimize water losses.

The report also reviewed the physical characteristics of the basin, its economic conditions and trends, and its water problems, and presented a recommended program of projects to put the co-ordinated plan of water development into action.

This is perhaps the first comprehensive program of river drainage basin improvement carefully developed in the United States, with three State groups participating and with competent engineering control and direction. It offers for the first time a skilfully drawn program for water conservation, susceptible to step-by-step construction in accordance with the financial capacity of the territory to carry the load. It provides for flood control in heavy run-off periods and for low-water supplementation in dry years.

The findings of the report are now (1937) being put into effect by local groups, and where money is being spent by Federal agencies in the territory, it is being spent in accordance with the definitely formulated plan.

The development of such a program implies time and the availability of money. It cannot be accomplished otherwise. The case in point illustrates, however, that it is feasible, with a certain amount of central direction and stimulation, to interest individual States in the preparation of a joint comprehensive program of action. It is hoped that this precedent may be followed with necessary local modification by other groups of States having similar pressing problems of water use. It is not implied, of course, that local pressures may not create deviations from the master plan, but the existence of an agreed master plan generally reduces these hazards to a minimum.

The Rio Grande.—Competitive uses of the water of the Rio Grande had reached such an acute stage in 1935 that it became necessary for the President of the United States to issue an order preventing the release of further Federal funds for any projects on this basin north of El Paso, Tex., until the National Resources Committee had approved the undertaking. An interstate compact between the States of Colorado, New Mexico, and Texas had been in operation for some time and was to expire in June, 1937.

In view of the conflicts of interest these three States, together with the Federal agencies concerned, undertook a detailed study of the water resources of the basin. Both Federal and State groups have contributed money and personnel to the undertaking. The studies should disclose non-controversial bases for the development of new agreements. They may be substitutes for Court action and may lead to the evolution of interstate agreements for the use of the Rio Grande water without basic controversy. The method of approach is again one for special purposes which might be adopted in other areas. Again, time and money are pre-requisites; it should be emphasized that without these two elements programs and policies cannot be "pulled out of thin air."

Kansas River Flood Studies.—The first contribution to the study of the Kansas River flood problem by the Water Resources Committee, or its predecessors, was made on November 1, 1934, when $25 000 was made available for expenditure under the direction of the Chief of Engineers, U. S. Army, for survey and investigation of the Milford Reservoir site on the Lower Republican River and the Tuttle Creek site on the lower part of the Big Blue River. Before investigations were started, it was decided to hold up the work pending agreement with the Flood Protection Planning Committee for Greater Kansas City, in regard to the conduct of the study, and the appointment of a Consultant by the Committee. The agreement was soon reached, and the surveys went forward during the summer of 1935, with F. H. Fowler, M. Am. Soc. C. E., as the Consultant.

It shortly became aparent that a complete plan for reservoir control of the Kansas floods might require the construction of additional storage on tributaries entering the main stream below the Republican and the Big Blue, and that considerable information regarding flood conditions in Kansas City might be secured from a river model. On the recommendation of the Water Resources Committee additional funds were made available to the U. S. Division Engineer on November 27, 1935. These funds consisted of $22 500 for surveys and borings at supplementary storage sites, and $12 000 for a river model, which was later constructed at the U. S. Waterways Experiment Station, at Vicksburg, Miss.

During the spring of 1936, the Bureau of the Budget rescinded $8 000 of the $22 500, thus preventing temporarily the completion of the schedule of boring at the dam sites. However, shortly afterward, Congress authorized a preliminary examination of the Republican and Smoky Hill Rivers with a view to the control of floods, and the Flood Control Committee of the House of Representatives authorized the Corps of Engineers to review the Kansas River report already published[3]. As a result, $150 000 was made available to the U. S. Division Engineer for additional studies. Of this amount, $8 000 was used to replace the amount rescinded by the Bureau of the Budget. Two series of tests on the river model were completed at Vicksburg during 1936. The results have gone far toward remolding public opinion.

[3] H. R. Doc. 195, 73d Cong., 2d Sess.

Study of an alternative arrangement of levees, combined with bridges, was initiated by the U. S. District Engineer Office, in Kansas City, after a joint inspection by the U. S. Engineer and the Flood Control Committee in May, 1936.

The development of the final plan should represent the reconciled viewpoint of all interests. The method of approach illustrates the merit of joint review by the Corps of Engineers and the local Flood Control Committee and its engineer advisers.

Potomac River Conservancy District.—The Water Resources Committee in 1934 recommended to the President the establishment of a Potomac River Conservancy District, primarily for the purpose of providing a field laboratory for evolving engineering, administrative and financial principles, and methods of control. In this instance, as in others, the district was to be under the supervision of the interested States, with Federal participation in a minority membership on the commission. Financing of the enterprise was to be largely by local agencies, public and private, with the stimulation of the Federal Government. Its purpose is solely to develop methods of arriving at solutions on major drainage basins. No action had been taken on this proposal, as of March, 1937, but it is presented herein as another device for supplying the nation with additional information on the problem of drainage-basin control and development. It was proposed as a more practical alternative to nation-wide drainage basin authorities, for which the ground is not yet prepared and which only future careful study can demonstrate to be valid or invalid structures for drainage-basin operation.

The National Drainage Basin Study.—Most civil engineers are familiar with the drainage-basin study of the Water Resources Committee. Its purpose is to indicate the outstanding water problems in the various drainage areas of the country, to fit them tentatively into integrated patterns of water development and control, and to present, where existing data make it possible, specific construction and investigation projects as elements of an enlarged plan.

This study supplements and extends the work of the Mississippi Valley Committee of the Public Works Administration and the Water Planning Committee of the National Resources Board, which agencies have been succeeded by the present Water Resources Committee.

The study under the title of "Public Works Planning" was presented to the President by the National Resources Committee in December, 1936. The President forwarded it to the Congress of the United States on February 3, 1937. It is to be followed by the issuance of a volume containing the more detailed findings of the Water Resources Committee. The date of the publication of this latter volume is the early part of March, 1937.

For many basins, the plan contains relatively few construction projects, because of the lack of essential data. In others, fairly complete construction programs are already available. Obviously, none of the plans is either fixed or final. No one supposes that any long-term plan for any drainage area can be formulated, immediately, in detail, even if all requisite hydrologic data were available. The drainage-basin study, however, should result in a

significant contribution "to the framework of an enduring, but adjustable, national water plan." It should serve to arouse professional and lay interest in water resources integration, in the extension of these preliminary studies under local auspices, in emphasis upon multiple, rather than upon single-use, development of streams, and in relating water development more adequately to the entire economy of a basin.

Summary

In this paper, the writer has attempted to show that the development of a National Flood Protection Policy should not be separated from that of a National Water Resources Policy in general; that the elements of such policy are highly complex; that they run the gamut of engineering, finance, and administration; that they are bound up with the problem of public prejudice and pressure; that the contributions of engineering thought are the key to the ultimate evolution of a national policy; and that such engineering contributions must be made on the field of battle and not in *vacuo*. In order to clarify some of the issues and to evaluate some of the procedures, local and Federal public and private agencies are collaborating in certain studies. As time goes on the results of these studies should disclose certain principles of action and should supply the basis for an ultimate national policy, changing in character, logical in administration, and as free from prejudice and greed as is possible in a world still fortunately consisting of human beings. A perfect syllogism of action is probably not attainable.

THE ECONOMIC ASPECTS OF FLOOD CONTROL

By Nathan B. Jacobs,[4] M. Am. Soc. C. E.

Synopsis

Flood control is a problem of applied economics in which the social security and protection of life and property must be evaluated in balancing the annual savings from flood control against the cost of construction and maintenance. Floods damage the homes of those least able to bear the cost who only live in the path of the inundation because financially unable to move elsewhere. No formula has yet been developed to apportion costs of such works satisfactorily between Government (Federal, State, and Local), and property owners, but each plan represents a problem for solution not only in its engineering aspects but also in distributing the costs.

Introduction

Engineers are guided largely by the principles of economics. Rigid rules are followed to test the economic worthiness of buildings, bridges, dams, railways, etc., before capital is called upon to make the projects possible. Heretofore, proposals for flood protection have been subjected to the same rigid analysis. Measurable direct damage is estimated and tabulated. To this is added some evaluation of the indirect damages that cannot be definitely determined. From this, the average annual damages incurred within and without the flooded areas, are determined. If the return on capital to be invested in protective works, plus depreciation, maintenance, and operating expenses, is less than the average annual damages sustained, the proposed project is considered to be on a sound economic basis.

It is true that flood control involves questions of economics, because floods destroy wealth and property, disturb the production and exchange of goods, and endanger lives and health. The questions, however, are not so much those of pure economics as applied economics—the science that seeks the improvement of existing conditions and the translation of such knowledge into action. Should a strictly monetary evaluation be the only basis of providing relief from inundation by waters gone wild?

Social Security a Factor in Evaluating Flood Control

Widespread human suffering and appalling loss of life have a stronger universal appeal than the monetary and property damage suffered by industry, trade, and transportation in major catastrophes. As far back as 1882, President Chester A. Arthur, in a message to Congress, stressed the sufferings of the people of the Mississippi Delta. A former Governor of Louisiana, the Hon. John M. Parker, felt the principle of humanitarianism to be so

[4] Pres., Morris Knowles, Inc., Pittsburgh, Pa.

important that he maintained it to be inhuman to permit three-quarters of a million people to be driven from their homes and made dependent upon the charity of fellow Americans. In 1927, former President Herbert Hoover, Hon. M. Am. Soc. C. E., then Secretary of Commerce, in discussing the 1927 Mississippi flood, said the losses sustained touched the entire nation, and subtracted something from the wages or income of every citizen.

The concept of humanitarianism is forcibly brought to public attention by the wording of the Omnibus Flood Bill, enacted into law in 1936 by the 74th Congress of the United States. This law reads, in part:

"That the Federal Government should improve or participate in the improvement of navigable waters or their tributaries, including watersheds thereof, for flood control purposes if the benefits to whomsoever they may accrue are in excess of the estimated costs, and if the lives and social security of people are otherwise adversely affected."

It will be noted from this passage that as the law now stands there are other considerations to be reckoned with than money or property damage. Under this Act, it is proper for the Federal Government to participate in flood-control projects if the life or social security of the people is in danger. This criterion will be as controlling and persuasive as the financial formula by which it formerly was necessary that projects be economically justified on the basis of past experience. There is inculcated in the minds of this generation, more so than in those of the past, a higher regard for general welfare and human protection.

The construction of flood-protection projects on a monetary economic basis alone is a thing of the past. Where the lives of human beings are endangered and where great suffering may occur as a result of floods, protective works should be constructed. It is too much to expect that people shall move from the path of danger.

In 1906, there occurred one of the worst catastrophes in the history of the United States—the San Francisco earthquake—which rendered 25 000 homeless and caused monetary losses estimated at $600 000 000. The people of that city had the courage to rebuild on the same site, with the knowledge that no man-made protection could be afforded. Human nature is the same the world over, and cities ravaged by floods have been rebuilt just as San Francisco was. It seems to be the spirit of some to reside in danger zones. The fact that floods occur again and again has not acted in the past, and will not act in the future, as a deterrent to the rebuilding of homes and the re-establishment of commercial enterprises and industries in perilous localities. Since it is possible to provide measures of protection against floods, the people who live in flood-plains must be afforded that protection.

Furthermore, the returning to homes, or to the location of former homes, after a flood disaster, may not be so much a matter of choice as of compulsion. City planners claim that much of the territory that is inundated during a flood is not economically and practically adaptable for homes, and that, therefore, the proper procedure is to relocate these families on ground reasonably above the flood-plain. In large cities, however, many of the commercial and industrial enterprises are located as near the river as possible,

often for the purpose of obtaining water for condensing or cooling purposes; and many workers live in homes near the industries because of the convenience and economy of being able to walk to work in a short time. Although transportation is now much more readily available and more economical than formerly, the workers continue to live in these houses, possibly because there are no other houses available in the district.

For example, of the area in Pittsburgh, Pa., that was inundated in the flood of March, 1936, about 80% is zoned as "heavy industrial." Under the provisions of the zoning ordinance, no residence may be built in a heavy industrial district. Of course, the law is not *ex post facto*, and residences existing in such a district at the time of its passage (1923) were permitted to remain. However, it may be assumed that no new dwelling has been constructed in the heavy industrial zone since that time, and, further, that no house has been subject to major repair, because permits could not be obtained for either purpose. Many of the houses in the district have already been condemned by the Pittsburgh Housing Association as not fit for human habitation. During the flood, the Association made provisions to prevent tenants from returning to these condemned houses after the water receded; but as homes within the rent range of these tenants were not available in other and higher parts of the city, it was necessary, in order to re-house them, to allow them to return to their original homes. Such a condition may be alleviated by additional housing and proper city planning and zoning, but it cannot be cured.

Engineering Aspects

The economic problem of flood control does not have a single answer, such as the solution of an algebraic equation. There are many ways to reduce flood damages. Retention basins, storage reservoirs, levees, channels, and dikes are the common structures provided. The problem is not so much one of design and construction as of determining the most economical type of protective works. This requires a competent and comprehensive engineering study. A comparison of the initial cost of construction of the various plans on the same price basis is necessary. Annual charges must be compared against the average annual benefits to be derived, in order to determine the most economic project. Consideration must also be given to the degree of protection afforded by each plan, as the human and social welfare angle must not be ignored. It is also essential that those who must pay for the protective system have full knowledge of the cost, of what burden they must bear, and of what security the final developed plan provides.

The physical characteristics of the up-stream basins will have considerable bearing upon the plan to be selected. In some localities, sites for retarding dams may not be available. In others, the construction of the dams may be economically impossible; good borrow-pits may not be available for the construction of earth dams; and concrete masonry structures might prove to be far too costly. If the river traverses wide, flat lowlands, levees and channel improvements may be the best solution. In the adoption of any flood-

control plan, it should be emphasized that floods cannot be prevented. Protection against all floods will not be furnished by a control system; no one can predict the intensity of the maximum flood to come, but the damage caused by floods of certain heights can be reduced or entirely prevented, and that fact should be stressed by the promoters and designers of projects.

In the matter of design and operation of works to impound flood waters, there are two schools of thought. One holds that such reservoirs should act solely as retention basins, and be automatic in control. Others advocate reservoirs that may be used for many purposes, such as storage for power and water supply and for regulation of the stream during periods of low flow. The degree of flood protection desired establishes the basis for the selection of retention, or multi-service, reservoirs. Multi-service reservoirs may not provide maximum flood control, because rapid and heavy run-off may occur when they are full or nearly so. One or more of the services to be rendered by this type of reservoir must suffer because of the others. On the other hand, if automatically controlled retention basins are constructed, future conservation of water for other uses will be seriously hampered.

Examples of both methods of control are in service. The automatically controlled retention basin is best typified by the reservoirs in the Miami Conservancy District of Ohio. The more recently established Muskingum Watershed Conservancy District, in the same State, provides for protection against floods, by three automatic retention basins and eleven multi-service reservoirs. The Sacandaga Reservoir, constructed in 1930, in New York State, is an example of the multi-service type. It is operated as a storage reservoir to regulate the flow for power interests down stream. At the same time, it acts as a retarding basin to some extent, thereby lessening flood heights of the Hudson River, at Albany and Troy, N. Y.

Another typical multi-use reservoir is the one on the Tygart River, in West Virginia, which was designed primarily to aid navigation on the Monongahela River, but which can and will be used for flood-control purposes as well. It is planned to operate this reservoir so that it is empty near the end of December; from then until April 1, a volume of 100 000 acre-ft of water will be allowed to accumulate. This volume will be drawn on as required during the remaining nine months of the year to assist navigation. Flood water will be impounded during January, February, and March, thus reducing flood heights on the Monongahela River.

The Pymantuning Reservoir, on the Shenango River, in Pennsylvania, was designed primarily for low-water control and for recreational purposes. Since its completion in 1933, however, it has been responsible for lowering flood heights at Sharon, Pa., and other communities. (The reservoir was completed in June, 1933; the project was completed and the gates closed on January 23, 1934).

The first and foremost thought in providing flood-protection projects should be to obtain maximum security. It is conceded that many multi-service projects are worthy; but if flood protection is the major problem of the community, it should not be sacrificed to the benefit of other uses. On the

other hand, if the amount of protection desired permits the use of multi-service reservoirs, these projects should be erected, in order to further the conservation of water as a natural resource. Such reservoirs could be constructed initially for flood control, with provisions for the future installation of power units where they appeared feasible.

The Distribution of Costs

The furnishing of needed funds for flood-protection works is a controversial question. Should flood protection be considered a problem of local interests alone, or should Municipal, State, and Federal Governments share in the cost? Those who live within the flood path do not consider the cost of protective works to be a local problem. It is their contention that the entire cost, or the greater part of it, should be borne by governmental agencies outside the affected area. On the other hand, those who are not directly affected object strenuously to contributing to the cost. Flood disasters, however, have far-reaching effects and, indirectly, the entire nation suffers to a greater or less degree.

Following severe and damaging floods, there generally occurs in the inundated area and surrounding territory a depreciation in property values because of the slowing up of business activities. This is due to the hesitation of industrial organizations and individuals to locate in the community. Losses occur outside the flooded area through the disruption of electric service, water supplies, communication, transportation, and other utility services. The commodity market is disturbed, and prices rise. Counties, towns, and cities are losers to the extent that municipal property is damaged, and also to the extent that depreciation of private property reduces the value upon which taxes can be assessed Floods also cause a loss to the Federal Government, especially where navigation is affected; lock-gate machinery is damaged, erosion takes place around wing-walls, and costly re-dredging of channels is necessary, where silt and débris have been deposited by excessive flows.

The distribution of the cost of a flood-control project among the beneficiaries is a difficult problem. Should the Federal Government bear the entire cost, one-half the cost, or a lesser amount? What should be the share of State and Local Governments, and how should private interests and those immediately affected contribute toward the cost?

Flood-control works have been financed in various ways. The Pymatuning Reservoir, previously referred to, was paid for entirely by funds appropriated by the Commonwealth of Pennsylvania, except for certain land and flowage rights in Ohio, which were acquired by the industries and other interested parties in the Shenango and Beaver Valleys, and donated to the Commonwealth. No doubt the legal difficulties created when one State acquires property in another, prompted this action. In the case of the Sacandaga Reservoir, in New York State, almost 95% of the cost was assessed directly against the water-power properties benefited by the river regulation. Although the project has been of value to navigation and has provided flood protection, only 5% of its total cost has been borne by the public in general.

The Miami Conservancy District System was constructed at a cost of approximately \$30 000 000, without Federal or State assistance. This entire investment will ultimately be repaid by the interests benefited, about one-half by direct assessment on property in the district, and the remainder through tax levies. Of the total direct assessments, 5% is levied on farm lands, and 95% on city and industrial property.

The Muskingum Water-Shed project is being financed in a different manner. The total estimated cost of the work is given[5] as \$34 590 000. Of this amount, \$22 590 000 is to be furnished by the Federal Government from funds of the Public Works Administration for construction purposes. This contribution includes two elements: First, the PWA grant; and, second, the payment for Federal benefits. At the time the project was begun, the estimated construction cost was \$22 000 000. On this basis the PWA grant (30% of construction cost) was \$6 600 000. However, a study was made of the navigation benefits and the savings in future construction work for navigation projects on both the Muskingum and the Ohio Rivers. This benefit was due to the stabilization of the stream flow. Credit was also given for the lesser interruption to river traffic at times of flood level. Federal participation, due to the reduction of flood crests and the benefit to navigation, was computed at \$15 990 000, making the total Federal contribution to the project \$22 590 000. The State of Ohio participated on the basis of the protection afforded State property, including highways. In regard to the latter, it estimated that the benefits would be greater than the cost of the relocations made necessary by the construction of the reservoirs, and, therefore, permitted the Highway Department to co-operate with the Conservancy District in the relocations. The State Legislature further made a direct grant to the District of \$2 000 000. The remainder of the fund, comprising the total cost of approximately \$34 500 000, is being made up by the District itself through assessments on benefited properties, and taxation in the District.

The Miami Conservancy District is a more heavily industrialized area than the Muskingum District, and, therefore, it no doubt could bear better the burden of providing the entire cost without aid from State or Federal Government. Conditions exist throughout the country, however, where the affected area will not be able to bear the entire cost of protective works. This is especially true in agricultural areas.

In making an equitable distribution of the cost of flood-control projects, both direct and indirect benefits must be estimated. The total cost should then be apportioned among the various interests in accordance with the benefits. The percentage of the cost to be borne by each group cannot be set by any given formula; it should be determined in each individual case on the basis of the problems involved.

The formula for Federal participation in the cost of flood control, as set up in the Flood Control Act of 1936, is sufficiently discussed by Lt.-Col. Covell.

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[5] "The Muskingum Flood Control Work": A Symposium, *Civil Engineering*, January, 1936, p. 1.

Whatever may be the basis of cost distribution, it is essential that the full cost of the project be established. It should also be broken down to reveal the cost of land, rights of way, structures, equipment, and all other major elements, not only so that these costs may be distributed among the beneficiaries, but also to permit the etablishment of correct depreciation rates on the various projects.

Benefits of Flood Control

Alleviation of major floods will undoubtedly improve the productive efficiency of the affected areas. When all industry and trade are stopped by flood, it requires weeks, or months, to restore them to normal operating capacity. While repairs are being made, disrupted organizations cannot concentrate upon their normal activities. Their efficiency is lowered, with the result that not only they, but their clients and customers also, suffer to some extent.

With flood-control systems installed, rail and river transportation systems will be free of their worst enemy, and the keeping of transportation systems in a state of efficient operation not only benefits local interests in the flooded area, but also the nation as a whole. Particularly would this be so in case of war. Had the 1936 Pittsburgh flood or the 1927 Mississippi Valley flood occurred in time of war, carefully timed shipments of supplies and materials to the seaports would have been seriously delayed. If the country had been involved in repelling an invasion, the hampering effect of the floods on the efficiency of the military organization might have been even more serious.

The highway system is just as important as the railroad and navigation systems, and flood-protection work will enable the construction of more dependable highways throughout the flooded areas. Airways, too, will be benefited in many cases by the possibility of establishing terminals nearer to the centers of large cities.

If transportation systems do not suffer recurring flood damages, maintenance charges will be reduced, and the money thus saved will be diverted to other and more productive channels.

Floods have done a vast amount of damage to agricultural lands. Whenever crops are destroyed, the loss is felt not only by those directly damaged, but by all who buy the products of agricultural industry. With less soil erosion in these agricultural districts, the farmer will be able to produce his crops without fear of their being washed away; and the improvement in his economic status will be a gain not only for him but for the entire community.

One of the most serious consequences of a disastrous flood is that, in so far as residential properties are concerned, it affects adversely those people who are least able to bear the burden. As previously stated, 80% of the property in Pittsburgh inundated in the 1936 flood was "heavy industrial", but thousands of people lived in this territory. They had not the wherewithal generally to move into better districts. Much of their income must go for replacing and restoring damaged properties. Incidentally, this reduces the demand for consumer goods, particularly food and raiment, and thereby

creates further economic maladjustments. Most of this could be avoided by proper provisions for flood control.

There are many other benefits of flood control that might be mentioned here, the danger of interruption of the water supply of river towns will be materially reduced. The value of industrial sites along the river plains will be increased, and better housing facilities can be constructed in their vicinity. Recreational facilities, such as parkways and parks along rivers, can be developed. Rural communities and business enterprises which depend upon tourist trade will profit through the prevention of stream erosion. The artificial lakes created by multi-service reservoirs will themselves provide a source of outdoor enjoyment that is rapidly becoming an important part of American life.

FLOOD CONDITIONS IN NEW ENGLAND

By W. F. Uhl,[a] M. Am. Soc. C. E.

Synopsis

In New England there have been two disastrous floods within the past nine years. The losses due to the flood of November, 1927, are estimated at about \$40 000 000, with nearly two-thirds of this in Vermont.

With the estimates virtually complete (February, 1937), the damages due to the March, 1936, flood amount to about \$70 000 000, of which \$55 000 000 occurred in the Merrimac and Connecticut River Valleys alone.

The problem of preventing or lessening of damages to all kinds of property during floods has received consideration throughout the ages and many solutions have been proposed.

There is no complete practical solution but numerous partial solutions can be offered. Many of these are suitable only where peculiar conditions make them practical. Even where there is a practical solution, it may be not be economically feasible.

In New England, one of the methods which appears economically feasible is to build reservoirs designed for storage, which afford a large measure of flood protection as a by-product.

The statistical data collected on the 1927 and 1936 floods have materially increased the information available on the magnitude of flood flows.

Introduction

Most of the older cities situated on the rivers of New England have developed their present facilities and industries during the last 100 yr. Many of the important water-power centers have experienced their complete development during this period. Previously, floods, although undoubtedly causing much inconvenience, did little damage, as indicated by a lack of historical data regarding them.

Under existing conditions, with large and important cities and their various utilities situated in the river valleys, the flood hazard is becoming more serious and the economics of flood control are being investigated.

It is unfortunate that the records of the behavior of New England rivers are so meager, and it is of increasing importance that all available information be collected, recorded, and properly interpreted for future use in connection with possible mitigation of flood hazards.

[a] Hydr. Engr., Chas. T. Main, Inc., Boston, Mass.

Topography of New England

The topography of New England is dominated by the two northern ranges of the Appalachian Mountains, the Green Mountains, which practically divide the State of Vermont in two, and then extend across Western Massachusetts into Northern Connecticut as the Berkshire Hills; and the White Mountains, which occupy most of Northern New Hampshire and extend half-way across Northwestern Maine. The general level of these mountains is between 1 000 and 2 000 ft, with some areas above 2 000 ft and the mountain peaks considerably higher.

Five of the largest New England rivers rise within a 50-mile radius in the White Mountains: The Kennebec (drainage area, 5 970 sq miles), the Androscoggin (3 470 sq miles), the Saco (1 720 sq miles), the Merrimac (5 015 sq miles), and the Connecticut (11 320 sq miles). Speaking very generally, each of these water-sheds may be divided into three parts, as follows:

(1) An upland area, where the rivers move sluggishly through plateaus containing a large number of lakes. Valuable storage has been created in many of these lakes by the construction of dams at their outlets. The Rangeley System, on the Androscoggin River, and the Moosehead Lake System, on the Kennebec River, are the most notable examples of this.

(2) A central part, characterized by steep slopes and narrow valleys on the main rivers. Many of the tributaries entering this stretch come off the mountain sides, and are very flashy.

(3) A lowland area in which the slope of the rivers is less pronounced, but more concentrated by ledge barriers. Above these barriers, the valleys widen out, and the banks are low, so that in time of flood large amounts of channel storage space become available. It is at the ledge barriers that the valuable early water powers, such as Holyoke, Mass., Lowell, Mass., and Lewiston, Me., were developed. The tributaries in this stretch are generally sluggish, with practically all the available fall and some storage developed.

The Penobscot River (drainage area, 8 570 sq miles), the largest in Maine, rises in the mountains east of the head-waters of the Kennebec. Compared to the other Maine rivers the level upland part is larger and the water-shed wider in proportion to the length. Natural lakes and reservoirs are scattered over the entire drainage area.

West of the Connecticut lies the Housatonic River (drainage area, 1 930 sq miles), which rises in the Berkshire Hills and flows south across Massachusetts and Connecticut. This river is relatively flat, with a wide valley affording much channel storage in times of flood. The amount of developed storage is small.

The west slope of the Green Mountains is drained by the four Vermont rivers which are tributary to the St. Lawrence River by way of Lake Champlain: Otter Creek (the farthest south), the Winooski, the Lamoille, and the Missisquoi, the drainage areas being 925, 1 076, 710, and 885 sq miles, respectively. Otter Creek is the flattest and most sluggish, with some natural storage in the numerous marshes and flats. The other three streams to the north are more precipitous, with very little storage, either natural or developed.

THE 1927 FLOOD

The floods of November, 1927, and March, 1936, are outstanding events in the recent history of New England. The two floods, both of which

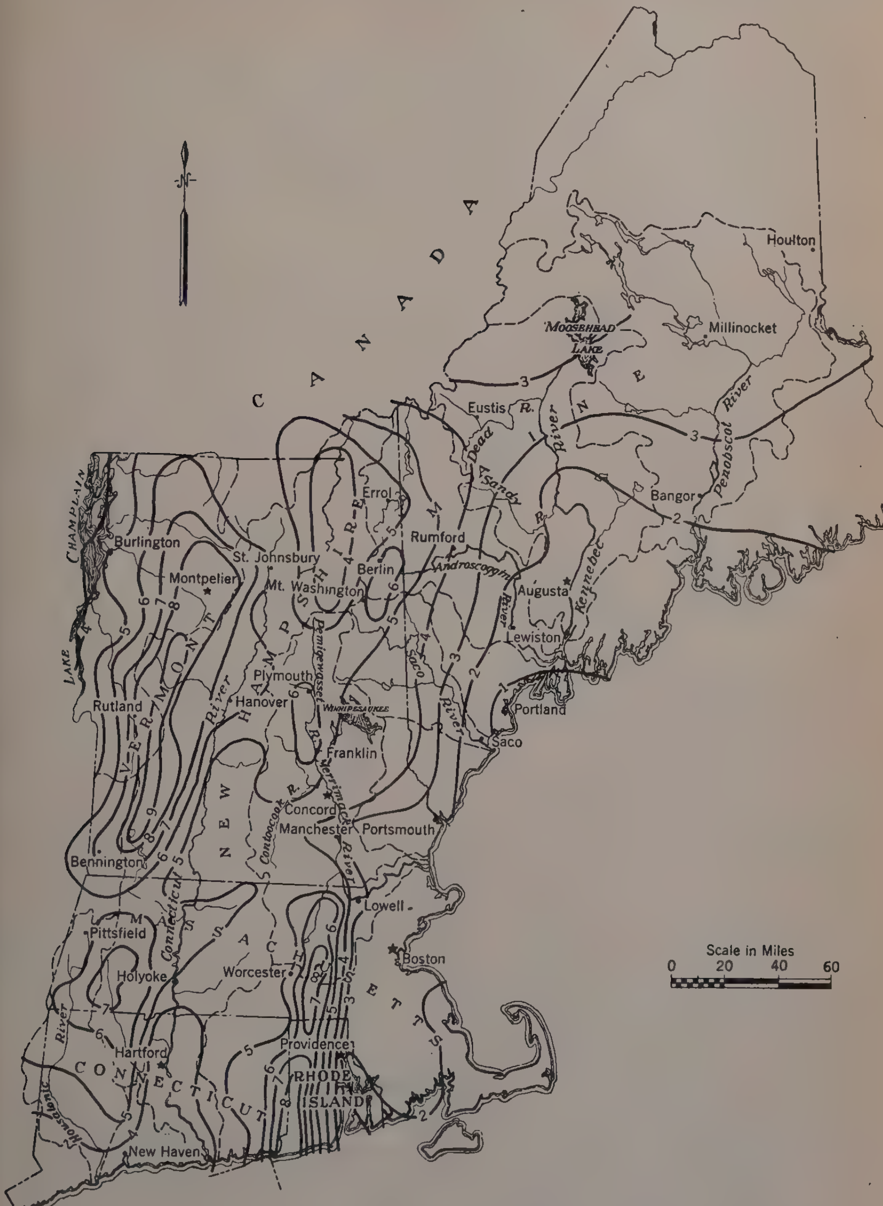

FIG. 1.—RAINFALL IN NEW ENGLAND, NOVEMBER 3 AND 4, 1936.

exceeded all previous records in their respective sections, were caused by altogether different climatological combinations.

The 1927 flood was much more localized than the 1936 flood. The worst of the damage was confined to the State of Vermont, with spectacularly high flows occurring on drainage areas of 1 000 sq miles or less.

On the other hand, the effect of the 1936 flood was felt over most of New England, the areas least affected being the State of Vermont and Northern Maine. It was on the large rivers that the record-breaking flows occurred. Although the total damage in the 1936 flood was about twice as great as in 1927, the devastation wrought by the 1927 flood was much more intense in the localities affected, and the loss of life was much greater.

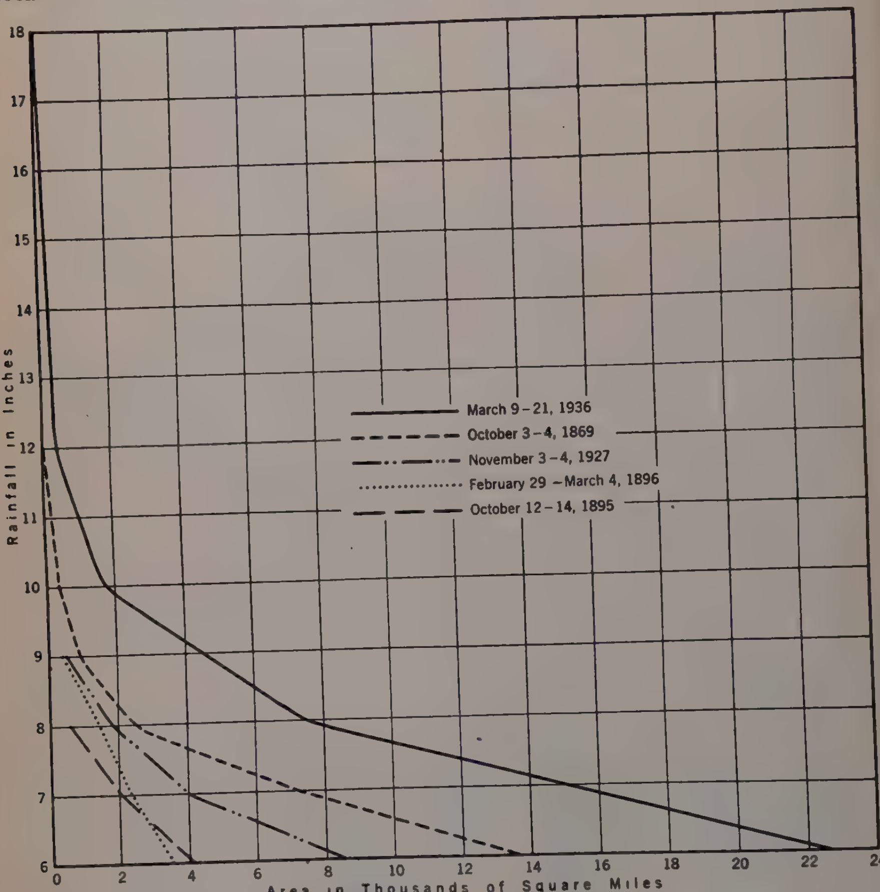

FIG. 2.—RAINFALL-AREA RELATION FOR GREAT STORMS IN NEW ENGLAND

The November, 1927, flood resulted from the torrential rains produced when a tropical storm, coming up from the South, was caught between and forced over two cold areas of high pressure, one moving in from the northeast and another from the northwest. When this storm began, the ground was still saturated from the heavy rains of October 18 to 21; the surface storage had been largely utilized; and the rivers had already risen to medium stages.

The rainfall which produced the 1927 flood occurred almost entirely on November 3 and 4 (Fig. 1). The precipitation was greatest in Vermont, with the maximum recorded at Somerset (Elevation 2 080), in the southwestern part of the State (9.65 in. in two days). It is probable that 11 to 12 in. fell in the higher portions of the Green Mountains. Another area of high rainfall was centered near the Rhode Island–Connecticut State line. The 2-day rainfall exceeded 9 in. over an area of about 500 sq miles, 460 of which was in Vermont, in the Green Mountains, and 40 in Rhode Island and Connecticut. The rainfall exceeded 5 in. over an area of about 22 000 sq miles, or one-third the area of New England. Data on the rainfall-area relations of this storm and other great storms in New England are plotted on Fig. 2.

As a result of this great precipitation, the rivers quickly rose to flood stages on November 3 and 4 and filled many of the valleys. Excessively high velocities on the rivers with steep slopes, such as the Winooski and the White in Vermont, caused sudden flood peaks which arrived at night. The inhabitants were taken unawares and many were drowned in their houses. In Vermont, 87 lives were lost, 55 of them in the Winooski River Valley. The total property damage in New England was about $40 000 000, of which $26 000 000 occurred in Vermont. This was equivalent to 3.27% of the taxable wealth of the State in 1920, or $74.30 per capita. About one-half the loss was concentrated in the Winooski Valley.

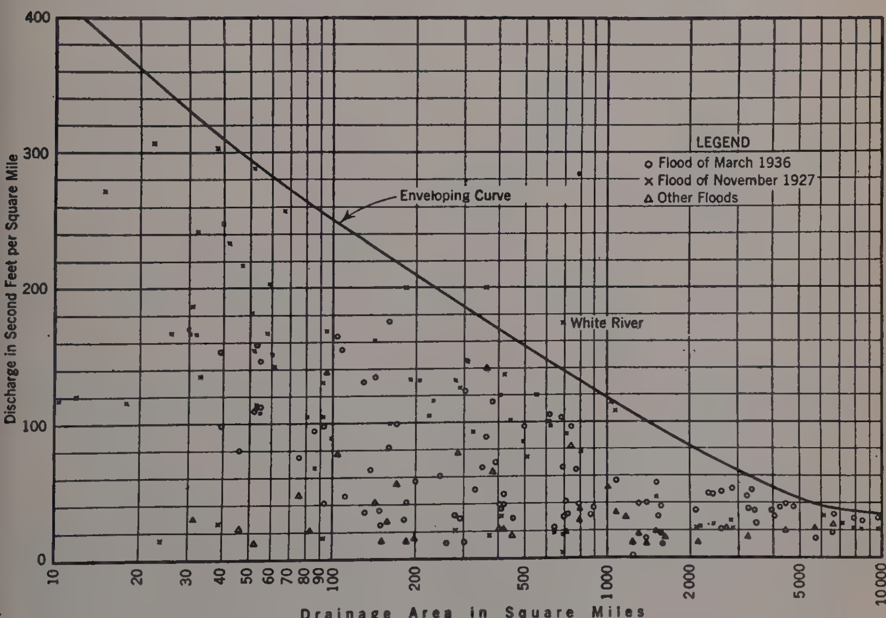

FIG. 3.—PEAK FLOWS OF NEW ENGLAND STREAMS

On the Winooski River, a flow of 116 000 cu ft per sec was observed, equivalent to 114 cu ft per sec per sq mile. On the White River, which drains the eastern slope of the Green Mountains and is tributary to the

Connecticut River, a flow of 140 000 cu ft per sec was observed, equivalent to 202 cu ft per sec per sq mile. These two flows are still (February, 1937) the maximum of record in New England for drainage areas of comparable size. On smaller drainage areas, much greater discharges per square mile were observed. Many of the 1927 flows are given in Table 3, and are plotted on Fig. 3.

The flood of November, 1927, has been described so thoroughly in the report of the Committee on Floods of the Boston Society of Civil Engineers[7] and by H. B. Kinnison, Assoc. M. Am. Soc. C. E.,[8] that further comment regarding it will be restricted to a comparison with the 1936 flood.

The 1936 Floods

The March, 1936, floods were due to an aggravated combination of the conditions that normally cause the annual spring freshets in New England. The fundamental cause was, of course, the abnormally high precipitation in a period of unseasonably warm weather which quickly melted the heavy snow cover. The first storm, March 12 and 13, melted most of the snow and produced flows considerably greater than the usual spring freshet. After a lull, with the rivers bank-full but receding, there came three more days of unusually heavy rain which fell on ground largely bare and frozen, with all the low spots full of water. Practically all of this second storm went rapidly into the streams and built up record-breaking flows. The excessive rainfall was the primary reason for the flood, but various other factors contributed largely to the eventual result.

Climatological Conditions Prior to the 1936 Flood.—In general, the winter of 1935–36 in New England was one of about average precipitation, but lower than average temperature. For New England as a whole, the average temperature for December, 1935, and January and February, 1936, was 20.6°F, which is 3.2° below the mean temperature for these months. This deficiency applied about equally to all six States.

The total precipitation for these three months was 10.32 in., which was 0.41 in. above the normal. The total snowfall, to March 1, as recorded by a number of stations, was 35 in. in the Southern States and 64 in. in the Northern States, or about 5 in. above normal.

The winter was a severe one, characterized by long-continued periods of sub-freezing weather, with few thaws. Data collected by the power companies and winter sports organizations indicate that by March 1 the snow cover in Northern New England was about 2.5 ft thick, with perhaps a foot more in the mountains; in Southern New England, it was from 1 ft to 2 ft thick.

During most winters the snow blanket in Central and Southern New England lies in layers of crust and powdery snow, due to the alternate periods of freezing and thawing. In the winter of 1935–1936, however, the snow came early, and because of the lack of thawing weather the blanket was light and powdery throughout its depth, which made it much more susceptible to quick run-off than usual.

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[7] *Journal*, Boston Soc. of Civ. Engrs., September, 1930.

[8] "The New England Flood of November, 1927", *Water Supply Paper 636-C*, U. S. Geological Survey.

From March 1 to March 10, conditions were generally normal, with one hard freeze on March 2. On March 3, there was a moderate snowfall, adding

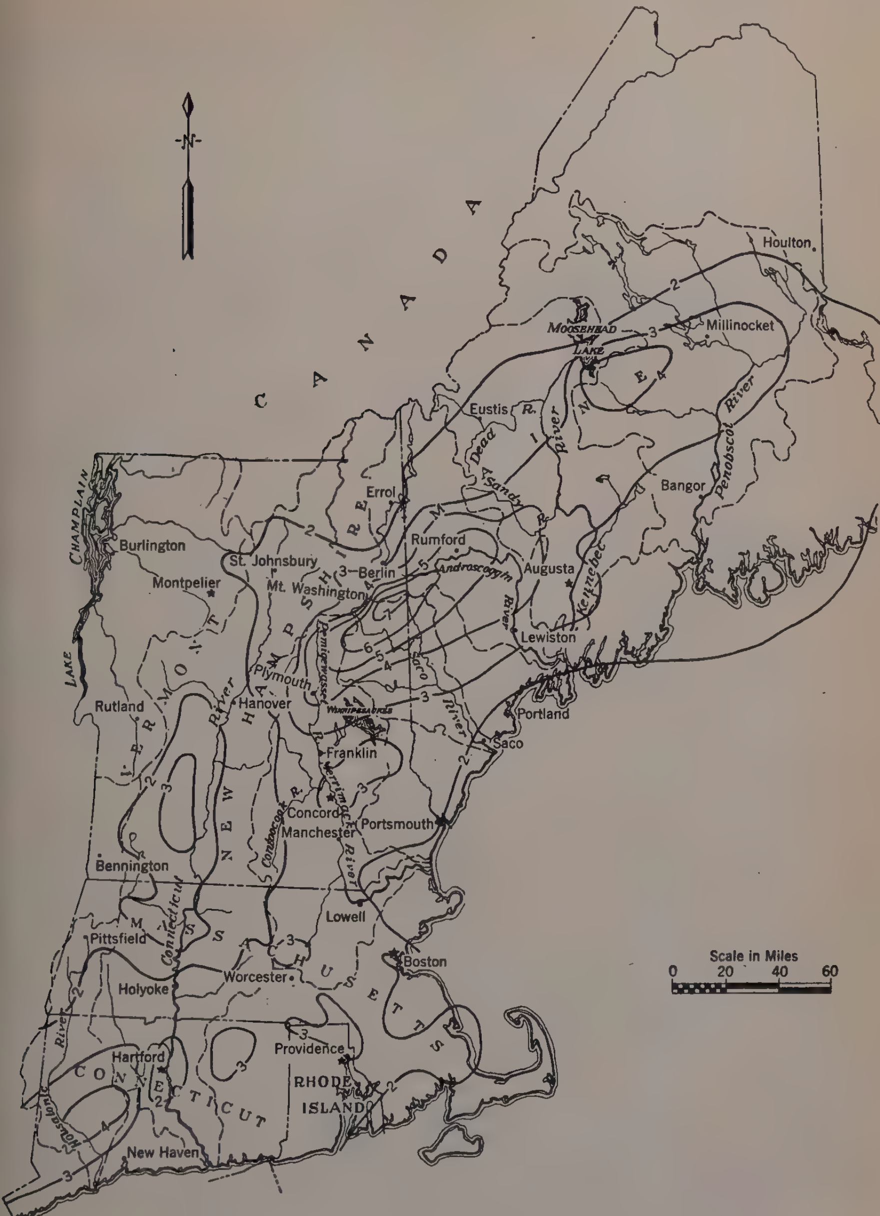


Fig. 4.—Rainfall in New England, March 11–13, 1936

a few inches to the existing snow cover. On March 9 a rise in temperature to above normal began, coupled with a light snowfall in Northern New England and moderate rain in Southern New England, on March 9 and 10.

TABLE 1.—CLIMATOLOGICAL DATA AT TYPICA

Drainage Basin	Penobscot					Kennebec					Androscoggin					Saco				
Station	Millinocket, Me.					Winslow, Me.					Berlin, N. H.					Pinkham Notch, N. H.				
	388 feet					90 feet					1 110 feet					2 000 feet				
Elevation	Precipitation, in inches	Snow fall, in inches	Snow on ground, in in.	Temperature, in degrees Fahrenheit		Precipitation, in inches	Snow fall, in inches	Snow on ground, in in.	Temperature, in degrees Fahrenheit		Precipitation, in inches	Snow fall, in inches	Snow on ground, in in.	Temperature, in degrees Fahrenheit		Precipitation, in inches	Snow fall, in inches	Snow on ground, in in.	Temperature, in degrees Fahrenheit	
				Max.	Min.				Max.	Min.				Max.	Min.				Max.	Min.
March 1..	....	..	34	28	19	....	..	..	33	0	....	..	30	27	2	0.02	T†	45.5	24	
2..	....	..	..	30	−12	....	..	..	30	4	....	..	30	28	—20	....	..	45.5	25	—1
3..	0.81	..	..	26	−7	0.53	..	18	30	6	T†	T†	30	33	—7	0.25	4.0	49	30	1
4..	....	..	..	39	23	....	..	..	45	21	0.04	0.4	28	48	23	0.01	T†	47§	45	
5..	0.05	0.5	..	36	29	T†	T†	18	41	21	....	..	22	34	25	T†	T†	47	44	2
6..	....	..	..	36	17	....	..	..	41	22	....	..	20	26	14	0:03	0.3	47	25	10
7..	....	..	42	28	−7	....	..	..	33	1	0.02	0.3	20	30	—12	....	..	46.8	24	—9
8..	....	..	..	34	−14	0.15	..	17	35	10	....	..	20	29	—8	....	..	46.5	30	—5
9..	0.06	0.8	..	33	1	0.24	..	..	36	12	0.19	1.5	22	35	17	0.86	7	48.0	32	17
10..	....	..	..	48	25	....	..	..	42	28	0.21	1.5	22	45	25	0.07	..	46.5§	44	24
11..	0.69	..	..	49	30	0.82	..	..	47	31	0.02	..	20	50	29	0.69	..	43.5§	44	38
12..	2.94	..	..	50	36	1.81	..	0	52	37	1.68‡	..	12	51	33	6.46	..	36	44	38
13..	0.05	..	..	51	36	....	..	..	50	35	1.01	..	P*	40	34	0.63	T†	34	43	36
14..	....	..	34	40	29	....	..	..	41	29	0.02	..	P*	37	25	0:08	T†	34.5	33	23
15..	0.19	..	..	41	24	....	..	..	50	25	T†	..	P*	46	21	T†	T†	33	42	18
16..	0.73	..	..	36	34	0.18	..	..	49	33	0.09	..	P*	46	33	0.86	..	32.5	37	32
17..	0.11	..	21	59	34	0.04	..	..	64	34	0.89	..	P*	53	35	0.44	..	31.0	45	33
18..	0.32	..	..	62	42	0.97	..	..	61	45	0.52	..	P*	54	44	6.27	..	24.0	48	42
19..	0.83	..	..	55	46	0.09	..	..	58	47	2.20	...	T†	51	46	4.05	..	20.5	46	36
20..	0.03	..	..	49	37	0.05	..	..	51	30	0.42	..	T†	46	35	0.05	..	18.0	40	33
21..	0.49	..	..	40	29	0.68	..	..	48	35	0,16	..	T†	45	31	1.33	..	16.2	46	32
22..	0.12	..	..	46	37	....	..	..	50	35	2.12	..	T†	38	32	0.64	1.4	16.5	38	30
23..	....	..	..	51	34	....	..	..	54	33	0.04	..	T†	45	29	0.04	0.4	15.5	48	27
24..	....	..	..	51	27	....	..	..	57	25	....	..	..	55	28	....	..	14	50	34
25..	....	..	..	43	29	....	..	..	55	35	0.12	..	T†	43	34	0.28	T†	13	40	32
26..	....	..	..	55	35	....	..	..	60	35	0.02	..	T†	55	32	0.01	..	11	52	30
27..	0.63	0.5	..	49	24	0.62	..	..	59	26	....	..	..	45	25	0.38	..	11	35	27
28..	....	..	..	50	32	....	..	..	54	35	0.57	..	..	49	34	0.34	..	10	44	32
29..	....	..	..	52	33	....	..	..	56	33	0.06	0.6	0.4	59	24	0.01	0.1	7	54	29
30..	0.01	..	..	65	31	....	..	..	69	33	....	..	..	66	34	....	..	3	60	35
31..	0.05	..	P*	62	38	....	..	..	65	39	0.06	..	..	58	45	0.06	..	0	54	28
Total.....	8.11	1.8	..	...	...	6.18	..	..	...	...	10.46	4.3	..	...	...	23.86	..	...	...	..
Average...	....	..	..	45.0	24.9	....	..	..	48.9	26.9	....	..	..	44.0	23.9	....	..	...	40.8	23
Mean.....	....	..	..	35.0		....	..	..	37.9		....	..	..	34.0		....	..	...	32.2	
Departure from normal.....	+4.52	..	..	+7.8		+2.90	..	..	+7.4		+7.39	..	..	+6.5		....	..	...	....	
Years of record...	37	..	..	34		41	..	..	32		37	..	..	28		7	..	...	7	

* P = Patches. † T = Trace. ‡ Hail during rain. § Snow settled.

TATIONS IN NEW ENGLAND, MARCH, 1936

rainage Basin	Merrimack					Connecticut					Housatonic					Champlain				
Station	Plymouth, N. H.					Cavendish, Vt.					Falls Village, Conn.					Northfield, Vt.				
levation	500 feet					800 feet					585 feet					876 feet				
	Precipitation, in inches	Snow fall, in inches	Snow on ground, in in.	Temperature, in degrees Fahrenheit Max.	Min.	Precipitation, in inches	Snow fall, in inches	Snow on ground, in in.	Temperature, in degrees Fahrenheit Max.	Min.	Precipitation, in inches	Snow fall, in inches	Snow on ground, in in.	Temperature, in degrees Fahrenheit Max.	Min.	Precipitation, in inches	Snow fall, in inches	Snow on ground, in in.	Temperature, in degrees Fahrenheit Max.	Min.
arch 1..	....	..	23	26	9	T†	T†	31	32	19	....	..	12	38	18	....	..	23	21	—3
2..	....	..	21	25	—14	0.00	..	31	30	-14	....	..	..	34	—1	0.03	..	..	29	—14
3..	0.25	20	22	34	17	0.27	3.0	33	33	21	0.12	2.0	..	43	25	0.10	1.3	..	33	20
4..	....	..	20	43	10	0.00	..	30	47	5	....	..	..	49	24	....	..	..	49	12
5..	....	..	20	35	20	0.00	..	29.5	39	29	0.15	1.0	..	45	26	....	..	..	43	22
6..	T†	T†	19	30	10	T†	T†	29.5	30	9	....	..	..	31	8	0.02	0.3	..	25	—2
7..	....	..	18	27	—9	....	..	29	25	—7	....	..	..	32	3	....	..	..	25	—7
8..	....	..	17	35	—11	....	..	28	38	—9	....	..	..	41	—1	T†	..	..	35	—6
9..	0.75	0.5	20	32	19	0.26	1.7	29	36	21	0.06	..	..	39	29	0.16	0.4	17	37	27
10..	T†	...	18	41	29	0.05	..	27	44	31	T†	..	..	59	32	....	..	..	45	31
11..	0.46	..	16	42	30	0.26	..	24	45	33	0.12	..	..	53	35	0.09	0	..	50	34
12..	3.46	..	13	46	36	2.66	..	16	46	35	1.80	..	..	53	37	1.12	0	..	49	35
13..	0.33	..	11	42	30	0.30	T†	12	42	32	0.44	..	..	48	26	0.09	0.1	..	47	28
14..	....	..	10	37	21	0.08	1.4	11.5	39	26	0.02	..	..	43	29	0.01	0.5	..	38	20
15..	T†	..	9	44	20	0.00	..	9	50	24	....	..	0	64	27	....	..	..	48	20
16..	0.49	..	8	40	30	0.19	..	8	40	32	....	..	....	60	34	1.34	0	5	41	35
17..	0.31	..	7	45	30	0.51	..	6	43	33	0.11	..	..	62	39	0.29	0	..	53	33
18..	2.12	..	4	50	40	1.52	..	4	51	37	2.34	..	..	60	46	0.59	0	..	55	48
19..	1.82	..	P*	48	40	1.03	..	2	49	36	....	..	..	58	43	0.18	0	..	54	40
20..	....	..	P*	46	33	0.00	..	1	50	35	0.02	..	..	60	36	....	..	..	44	31
21..	1.13	..	..	45	28	0.86	..	T†	47	31	0.05	..	..	58	39	0.90	0	..	44	30
22..	0.31	T†	..	39	32	0.17	..	T†	40	34	0.08	..	..	48	30	0.10	0.3	..	40	33
23..	....	..	..	50	29	T†	..	T†	51	28	....	..	..	55	35	....	T†	..	47	28
24..	....	..	..	55	25	0.00	..	T†	61	28	....	..	..	67	34	0.19	0	..	59	27
25..	0.27	..	..	48	33	0.39	..	T†	51	36	0.35	..	..	65	42	0.34	0	..	43	38
26..	....	..	..	58	26	0.00	..	T†	60	29	....	..	..	64	30	....	0	..	57	29
27..	0.45	..	..	41	24	0.78	..	T†	52	26	0.05	..	..	52	30	0.33	0	..	43	27
28..	0.27	..	..	52	33	0.05	..	T†	53	31	0.88	..	..	56	40	0	0	..	50	32
29..	....	..	..	63	30	0.00	..	T†	65	28	....	..	..	68	24	T†	0	..	63	28
30..	....	..	..	66	28	0.00	..	T†	68	38	....	..	..	72	34	0.01	0	..	64	33
31..	....	..	0	52	34	0.09	..	0	63	38	....	..	0	69	38	0.16	0	0	56	30
otal.....	12.42	2.5	...	...	..	9.47	6.1	..	...	...	6.59	3.0	..	...	...	6.05	2.9	..	...	....
verage...	....	..	..	43.1	23.0	....	..	..	45.8	24.8	....	..	..	53	28.9	....	..	..	44.7	23.8
ean	....	..	..	33.0		....	..	..	35.3		....	..	..	40.9		....	..	..	34.2	
eparture from normal.....	+9.21	..	..	+4.3		+6.31	..	..	+5.8		+3.42	..	..	+6.7		+3.49	..	..	+7.8	
ears of record...	49	..	..	49		34	..	..	34		46	..	..	20		....	..	..	50	

TABLE 2.—PRECIPITATION AND SNOW DEPLETION AT SELECTED STATIONS IN NEW ENGLAND, MARCH 9–21, 1936

Station	Altitude, in feet	Drainage area	Precipitation, in Inches, by Calendar Days in March													Total, March 9–21, 1936	Snow Depletion		Total precipitation + water equivalent of snow depletion, in inches
			9	10	11	12	13	14	15	16	17	18	19	20	21		Inches	Water equivalent, in inches	
Houlton, Me.	364	St. John River	0.04	....	....	0.91	0.65	0.07	....	0.37	0.38	....	0.40	0.09	0.15	3.06	29	7.25	10.31
Millinocket, Me.	388	Penobscot River	0.06	....	0.69	2.94	0.05	....	0.19	0.73	0.11	0.32	0.83	0.03	0.49	6.44	21	5.04	11.48
Eustis, Me.	450	Kennebec River	0.11	....	T*	1.61	0.23	0.11	....	0.61	0.80	0.48	0.65	0.10	1.20	5.90	31	7.76	13.66
Madison, Me.	257	Kennebec River	0.49	....	0.88	2.59	0.19	....	....	0.85	0.26	0.86	0.49	0.02	0.87	7.50	18	4.50	12.00
Lewiston, Me.	182	Androscoggin River	0.57	0.17	0.13	3.07	0.37	....	....	0.02	0.10	0.65	1.60	....	1.56	8.24	18	4.4	12.64
Mt. Washington, N. H.	6 270	Androscoggin River	0.96	0.70	....	1.66	2.27	0.21	....	0.14	1.15	1.32	2.53	0.94	1.01	12.89	19	4.75	17.64
Rumford, Me.	505	Androscoggin River	0.28	....	0.38	5.07	0.39	T*	....	0.81	0.26	1.60	1.29	0.45	1.36	11.89	18	4.50	16.39
Portland, Me.	0	Coastal	0.58	0.12	0.25	1.23	0.24	....	T*	0.03	0.03	0.22	0.73	....	1.17	4.60	15	4.00	8.60
Lincoln, N. H.	1 200	Merrimac River	0.50	0.49	....	2.21	2.49	0.01	....	0.20	0.91	0.19	1.23	0.40	0.05	8.68	29	7.75	16.43
Franklin, N. H.	390	Merrimac River	0.41	0.17	0.19	2.70	0.29	....	....	0.17	0.07	0.97	1.41	....	0.84	7.22	18	4.50	11.72
Wolfboro Falls, N. H.	514	Merrimac River	0.64	0.10	0.15	1.98	0.30	....	....	0.04	0.05	1.13	1.23	0.01	1.25	6.88	25	5.51	12.39
Haverhill, Mass.	50	Merrimac River	0.15	0.14	0.05	1.55	0.07	....	0.03	T*	....	0.44	1.12	....	0.99	4.54	4	1.00	5.54
First Connecticut Lake, N. H.	1 660	Connecticut River	....	....	0.25	0.79	0.03	T*	0.01	1.35	0.04	0.54	0.20	0.01	1.37	4.59	16	5.5	9.09
Bloomfield, Vt.	930	Connecticut River	0.16	....	....	1.14	0.27	0.02	....	0.82	0.61	0.10	0.04	0.23	0.53	3.92	31	7.75	11.67
Bethlehem, N. H.	1 440	Connecticut River	0.22	0.17	....	1.74	0.19	....	....	0.71	0.97	0.41	0.23	....	0.87	5.51	24	6.00	11.51
Hanover, N. H.	603	Connecticut River	0.40	0.02	0.06	1.31	0.10	0.02	0.01	0.39	0.41	0.77	0.25	T*	0.69	4.43	16	4.00	8.43
Newfane, Vt.	450	Connecticut River	0.20	0.05	0.50	2.47	0.20	0..0	....	0.05	0.70	3.45	0.20	0.25	0.75	8.82	15	3.39	12.21
Hoosac Tunnel, Mass.	800	Connecticut River	0.25	0.24	0.05	2.16	0.47	0.08	T*	0.03	0.25	1.19	2.31	0.16	0.55	7.74	18	3.54	11.28
Hubbardston, Mass.	1 030	Connecticut River	0.36	T*	1.43	0.20	0.01	T*	T*	T*	0.61	3.68	0.05	0.21	0.46	7.01	14	3.0	10.01
Collinsville, Conn.	300	Connecticut River	....	....	....	2.91	0.22	....	....	....	....	0.84	1.59	0	0.54	6.10	7	1.75	7.85
Stockbridge, Mass.	850	Housatonic River	0.05	T*	0.12	1.48	0.30	T*	....	T*	0.16	2.08	0.13	0	0.38	4.70	6	1.28	5.98
Cream Hill, Conn.	1 300	Housatonic River	0.05	....	0.52	1.53	0.36	0.06	....	....	0.16	2.18	0.26	0.03	0.41	5.56	12	3.00	8.56
Salisbury, Conn.	1 015	Housatonic River	0.09	....	0.23	1.54	0.40	....	....	....	0.18	2.25	0.21	0	0.47	5.37	10	2.50	7.87
Enosburg Falls, Vt.	422	Lake Champlain	0.15	....	....	0.78	0.45	....	....	0.37	....	0.45	....	....	0.54	2.74	9	2.25	4.99
Cornwall, Vt.	504	Lake Champlain	0.03	T*	....	1.55	....	....	....	0.68	0.75	....	....	0.32	0.45	3.78	16	6.41	10.19
Burlington, Vt.	403	Lake Champlain	0.03	T*	0.03	0.99	0.46	0.04	....	0.78	0.14	0.31	0.10	....	0.32	3.20	6	1.50	4.70

* T=Trace

Heavy rain began on March 11 and continued on March 12 and 13. The area of greatest precipitation centered in the White Mountains (see Fig. 4). The maximum rainfall in the 3-day period amounted to 7.78 in. and was reported at the Pinkham Notch Station, which is on the northeast slope of Mt. Washington, virtually on the divide between the Saco and Androscoggin water-sheds. In the same period, the water equivalent of the snow cover that melted in the vicinity of this station amounted to about 3 in. During this storm the rainfall was 7 in. over an area of about 70 sq miles, 6 in. over 450 sq miles, and 5 in. over 1 400 sq miles.

This precipitation was accompanied by an inrush of warm air from the Gulf, causing abnormally high temperatures which melted most of the snow cover in Southern New England and a part of that in Northern New England.

The daily record of maximum and minimum temperature, rainfall, and the quantity of snow cover at eight typical stations in New England are given in Table 1. The quantity and extent of the precipitation is also shown in Table 2, which gives the daily rainfall for twenty-six stations. Only those stations were included which had a record of snow depletion for the period. For each station the water equivalent of the snow depletion has been added to give the total equivalent precipitation during the period.

The combination of rainfall and melting snow resulted in a flood peak of major proportions on March 13, which carried a heavy ice burden. This flood is comparable to those which may be expected once in about every twenty-five years. The ice, being hard and strong, caused much damage to river structures, and in many cases to buildings located on the banks. Serious ice jams also formed on some of the rivers, causing severe valley flooding. In some instances the ice jams acted as diversion dams, causing new river channels to be cut.

The first flood had largely passed down the rivers by March 16, but some rain continued to fall afterward. Then a storm reached New England on March 18 and 19, giving rise to the record-breaking flood of March 19. This storm resulted from the junction of a moderately strong low-pressure area from the Gulf States with another low area coming from Indiana. The former caused an inflow of tropical and marine air which moved slowly to the north and east and reached New England on March 18.

In the second storm, March 17–19, inclusive (see Fig. 5 and Tables 1 and 2), the rainfall was much greater than in the first storm. The area of greatest precipitation was again the White Mountains, and the Pinkham Notch Station again reported the maximum precipitation, 10.76 in. The water equivalent of the snow cover which melted at this station was about 4 in., making a total equivalent precipitation of about 15 in. The rainfall was in excess of 8 in. over an area of about 300 sq miles, and in excess of 5 in. over about 1 000 sq miles. There was also about 5 in. of rainfall over 600 sq miles in Central Massachusetts not included in the areas mentioned in the previous sentence.

Another intense low-pressure area from the Ohio Valley followed on March 21, closely after the southern storm, causing heavy rains in certain sections

of New England on March 21, and a third flood peak of minor proportions on March 22 on the Merrimac River and some of its tributaries.

FIG. 5.—RAINFALL IN NEW ENGLAND, MARCH 17–19, 1936

The precipitation for the entire period, March 9 to 21, inclusive, was considerably greater than the sum of the precipitation during the two prin-

cipal storms (March 11 to 13, and March 17 to 19), as at many stations there was some rainfall almost every day. The total rainfall at Pinkham Notch was 21.79 in. The entire snow cover at this station, 30 in. (with an estimated rainfall equivalent of 7.5 in.), melted during the period, making a total equivalent precipitation about 29 in.

The general pattern of the isohyetal lines for the period is similar to that for the two individual storms. The area of greatest precipitation centered in the White Mountains near Mt. Washington and ran across the head-waters of the Merrimac, Saco, and west central portion of the Androscoggin water-shed in a roughly elliptical shape with the axis from northeast to southwest. The 10-in. isohyetal line encloses two large areas: One, of about 1 100 sq miles, in the White Mountains; the other, of about 600 sq miles, in the Androscoggin water-shed, with its center at Rumford, Me. The 10-in. isohyetals also enclosed a smaller area in Central New Hampshire, centering on the southeastern end of Lake Winnepesaukee, and another in Central Massachusetts, near Worcester. The extent of the rainfall for the period, March 9 to 21, inclusive, as compared with other great storms in New England, is shown on Fig. 2.

As one of the causes of the flood, the melting snow was next in importance to the rainfall. Data on the quantity of snow that melted, and on its water equivalent, are not nearly so complete as those on precipitation and temperature.

On the Connecticut River water-shed, an extensive survey of the snow cover and its water equivalent was made about March 1. Airplane views of the Connecticut Valley taken between March 21 and 24 indicate that by that time the snow cover was gone from the principal river valleys to within about 50 miles of the Canadian border.

On the Androscoggin River water-shed an excellent series of periodic observations of the snow cover and its water equivalent was taken in connection with the regulation of the storage. A similar set of observations, although not so extensive, was taken on the Upper Kennebec water-shed.

Scattered observations made in other parts of New England agree fairly well with the results in these three river valleys and indicate that, in general, except in Northern New England, the snow cover was equivalent to about 25% moisture; in other words, 4 in. of snow was equal to 1 in. of water. In the Upper Androscoggin and Kennebec River water-sheds the snow was heavier, with a higher moisture content. Observations made in this region about March 1, showed moisture in the layer of snow near the ground, indicating that the ground was frozen little, if at all, in this locality.

Fig. 6 is based largely on data from the stations listed in Tables 1 and 2. The lines show the sum of the rainfall and the water equivalent of the snow cover which melted between March 9 and March 21. The general pattern of these lines is similar to that of the isohyetals of Figs. 4 and 5. The 10-in. line includes about one-half of New England.

An examination of the climatological data for the entire month of March, 1936, shows the abnormality of the conditions which produced the floods.

Based on 98 stations, which cover the entire area of New England, the average temperature for the month was 38.9°, which is 6.5° above the mean for March for the past 49 yr, and higher than the average temperature for

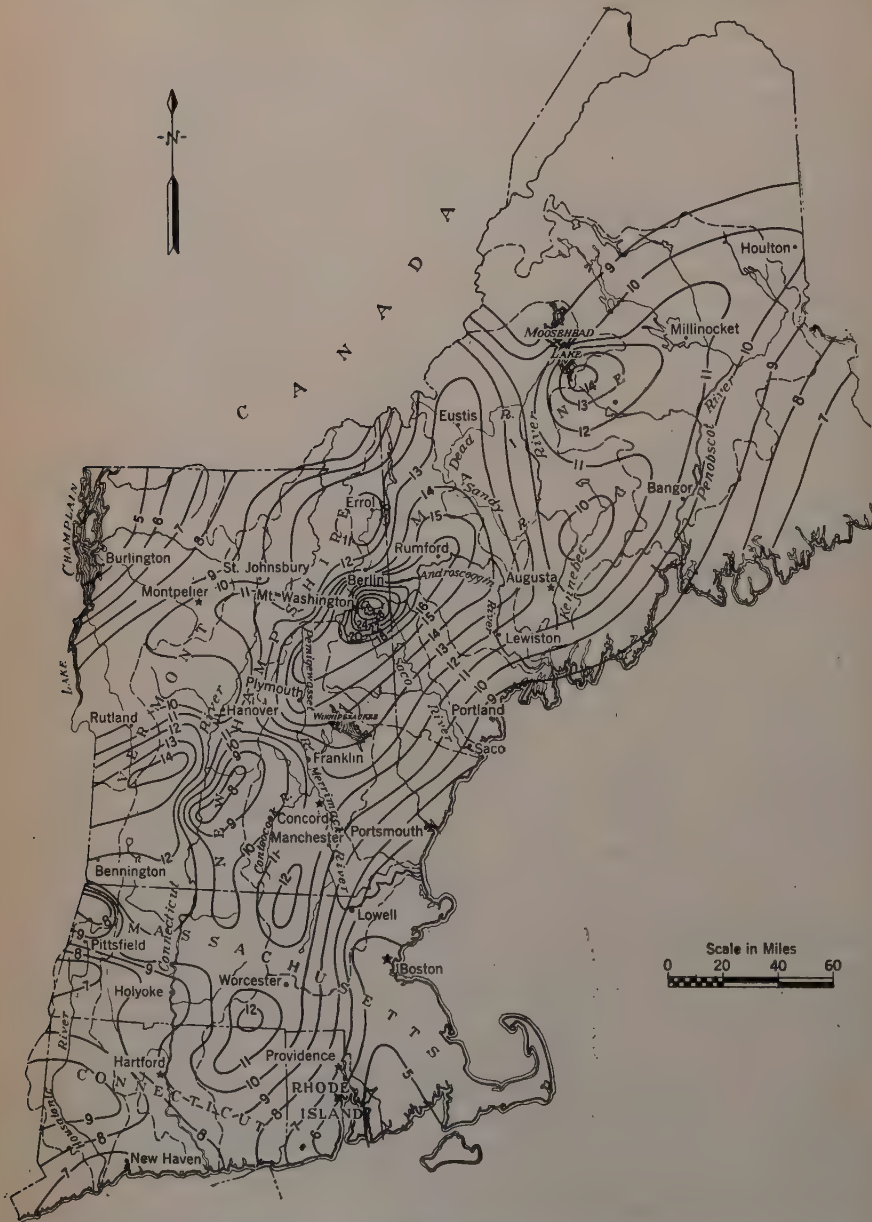


FIG. 6.—TOTAL RAINFALL PLUS WATER EQUIVALENT OF SNOW DEPLETION IN NEW ENGLAND, MARCH 9–21, 1936

March in every year of that period except 1903 and 1921. Based on 178 records, the average precipitation for March, 1936, was 8.04 in., which is 4.66 in. greater than the mean for the past 49 yr, and greater than any March on record, and most of this precipitation occurred during the period from March 9 to 21, inclusive.

Extent of the 1936 Flood.—In most of New England the 1936 flood exceeded all existing records for high flows. The areas in which the flows were not unusually great were Vermont and Northern Maine.

On most streams the flood had two peaks, the first reaching a maximum on March 13 and 14, and the second and greater flood reaching a maximum on March 19 and 20. On certain rivers there was a third peak much lower than the other two, on March 22 and 23. The first peak has frequently been called the "ice flood" because of the damage inflicted by floating ice.

More data are available on the 1936 flood than on any previous one. The 1927 flood made New England "flood conscious" and, in addition, before the 1936 flood, governmental and State agencies were actively studying flood control, water storage, and stream pollution. Furthermore, private agencies, the utilities, and the water power companies, have always been active in the collection of pertinent data. Thus, when it was evident that a major flood was happening, all these agencies united to obtain as many data as possible. As soon as the flood had subsided sufficiently to permit it, parties were put in the field to continue this work. Particular mention should be made of the U. S. Geological Survey force which, laboring under great difficulties, made current-meter measurements at or near the peaks at many stations, thus obtaining definite measurements of the higher flows and providing data for the extension of rating curves.

The 1936 flood peaks are given in Table 3, together with data on previously recorded maximums. This tabulation covers all the important rivers in New England, with special emphasis on those in which the 1936 flows were high. In addition, Table 3 includes the total run-off, in inches, where the data are available. The run-off has been computed in all cases for the period, March 12 to 26, inclusive, which seems to be the best average flood period on the various rivers. A consideration of the total run-off demonstrates the magnitude of the flow. The mean annual run-off of the larger New England rivers is about 22 in. In many cases, the total run-off for the 15-day flood period was as much as 9 in., or about 40% of this amount.

From the records of the U. S. Geological Survey and from those of the power companies, flood hydrographs have been plotted for significant points on the Kennebec, Merrimac, and Connecticut Rivers.

Kennebec River.—The hydrographs of the Kennebec River, in Maine (Fig. 7), cover the length of the main river, and are also well spaced as to drainage areas. The first flood peak, of March 13 and 14, was a "down-river" peak, and was produced by a moderate rainfall plus temperatures above freezing which melted a large portion of the snow cover on the lower water-shed. This peak was accompanied by ice jams from Bingham to tide-water at Augusta, and there were further ice jams on March 14 in the tidal part of the river all

TABLE 3.—NEW ENGLAND FLOOD FLOWS; 1936 PEAK DISCHARGES AND PREVIOUS MAXIMUMS

River and station	Drainage area, in square miles	Period of record	Maximum Discharge Previously Recorded: Date	Maximum Discharge Previously Recorded: Discharge: Cubic feet per second	Maximum Discharge Previously Recorded: Discharge: Cubic feet per second per square mile	1936 Peak Discharge: Date March, 1936	1936 Peak Discharge: Cubic feet per second	1936 Peak Discharge: Cubic feet per second per square mile	Run-off in inches, March 12–26, 1936, inclusive
St. John at Fort Kent, Me......	5 690	1926–	5/ 3/33	121,000	21.3	23	82 000	14.4	3.3
St. Croix at Baileyville, Me.....	1 320	1919–	5/ 1/33	23 300	17.7	23	17 100	13.0	4.9
PENOBSCOT RIVER BASIN:									
West Branch, near Medway, Me.	2 120	1916–	5/27/28	24 100	11.4	20	5 500	2.6	......
Penobscot at W. Enfield, Me....	6 600	1901–	5/ 1/23	153 000	23.2	21	125 000	19.0	6.1
East Branch at Grindstone, Me.	1 070	1902–	4/30/23	35 100	32.8	20	23 900	22.3	......
Mattawamkeag at Mattawamkeag, Me..................	1 500	1902–	5/ 1/23	43 900	29.3	23	30 400	21.7	......
Piscataquis near Foxcroft, Me...	286	1902–	9/29/09	21 700	76.4	20	19 200	67.2	......
Piscataquis at Medway, Me.....	1 170	1924–	9/18/32	35 000	29.9	20	58 400	49.9	......
KENNEBEC RIVER BASIN:									
Kennebec at Moosehead, Me...	1 240	1919–	4/ 9/29	13 600	11.0	3	2 280	1.84	0.2
Kennebec at The Forks, Me....	1 570	1901–	6/18/17	23 700	15.1	20	17 100	10.9	2.0
Kennebec at Wyman Dam, Me.	2 612	....	......	......	......	20	53 900	20.7	......
Kennebec at Bingham, Me.....	2 710	1907–1910 1930–	5/11/09	57 500	21.2	20	55 900	20.6	......
Kennebec at Skowhegan, Me...	3 950	....	......	......	......	20	133 000	33.7	......
Kennebec at Waterville, Me....	4 270	1902–	12/16/01	157 000	36.8	19	157 000	36.5	7.5
Dead at The Forks, Me........	878	1901–1907 1925–	4/30/33	23 800	27.1	20	28 400	32.4	8.5
Carrabasset nr. Northanson, Me.	351	1901–1907 1925–	9/17/32	20 100	57.2	19	23 700	67.5	12.4
Sebasticook nr. Pittsfield, Me...	598	1928–	4/16/34	9 400	......	22	15 000	25.1	......
ANDROSCOGGIN RIVER BASIN:									
Androscoggin nr. Gorham, N. H.	1 390	1929–	5/ 4/29	13 900	10.0	22	19 500	14.0	4.4
Androscoggin at Rumford, Me..	2 090	1892–	11/ 5/27	46 700	22.3	20	74 000	35.4	......
Androscoggin at Gulf Island, Me.	2 600	....	......	......	......	20	128 000	49.3	......
Androscoggin nr. Auburn, Me...	3 260	1928–	4/19/33	50 100	15.4	20	148 000	45.4	9.3
Androscoggin at Brunswick, Me.	3 430	....	......	......	......	21	121 000	35.3	......
Little Androscoggin nr. So. Paris, Me..........................	76.2	1913–1923 1931–	4/14/20	3 540	46.4	19	7 160	93.9	......
Swift nr. Roxbury, Me.........	95.0	1929–	4/17/32	13 000	136.8	19	10 500	110.6	......
Presumpscot at Sebago Lake...	435	1887–	......	......	......	22	2 240	5.1	......
SACO RIVER BASIN:									
Saco nr. Conway, N. H.........	386	1903–1910 1927–	10/ 3/07	24 000	62.2	19	45 000	116.5	16.0
Saco at Cornish, Me..........	1 298	1916–	5/ 2/23	23 000	17.3	22	43 400	38.1	11.7
Saco at W. Buxton, Me........	1 572	1906–1916 1919–	5/ 2/23	27 800	17.7	22	53 260	37.7	......
Ossipee at Cornish, Me.........	453	1916–	4/19/23	7 440	16.4	21	14 000	30.9	10.1
Ellis at Goodrich Falls, N. H....	55.4	....	......	......	......	......	8 150	147	......
Salmon Falls nr. Lebanon, Me..	147	1928–	4/19/23	3 550	24.1	19	5 500	37.4	......
MERRIMAC RIVER BASIN:									
E. Br. Pemigewasset nr. Lincoln, N. H......................	104	1928–	5/ 3/29	8 000	76.9	19	17 000	163.5	20.2
Pemigewasset at Plymouth, N.H.	622	1886–	11/ 4/27	60 000	96.4	19	65 400	105.0	15.4
Pemigewasset at Ayers Is., N.H.	686	....	11/ 4/27	62 300	90.7	19	71 400	104.1	......
Merrimac at Franklin, N. H....	1 507	1903–	11/ 5/27	67 000	44.4	19	83 000	55.2	9.8
Merrimac at Manchester, N. H..	2 850	1924–	11/ 5/27	60 300	21.2	20	144 000	50.4	10.4
Merrimac below Lowell, Mass..	4 424	1846–	4/19/52	104 000	23.5	20	173 000	39.1	9.9
Merrimac at Lawrence, Mass..	4 672	1880–	3/ 3/96	86 900	18.6	20	174 000	37.2	9.6
Bakers nr. Rumney, N. H.	143	1928–	4/12/32	5 280	40.7	19	13 100	133.5	18.6
Smith nr. Bristol, N. H........	85.8	1918–	11/ 4/27	5 800	67.5	19	8 150	95.0	16.6
Contoocook at Penacook N. H..	766	1928–	4/20/33	17 600	23.0	20	45 900	59.8	12.1
Suncook at N. Chichester, N. H.	157	1918–	4/ 7/23	4 300	27.4	19	12 800	81.5	......
Souhegan at Merrimac, N. H...	171	1909–	4/ 8/24	9 300	54.4	19	16 900	98.8	13.9
N. Br. Nashua nr. Leominster, Mass........................	107	....	......	......	......	18	16 400	82.3	12.1
Concord at Lowell, Mass.......	376	....	......	......	......	......	7 500	20.0	......
Ipswich nr. Ipswich, Mass......	122	1930–	......	......	......	15	2 490	20.4	7.5
Blackstone at Woonsocket, R. I.	416	1929–	11/ 5/27	12 900	31.1	19	15 000	36.1	9.1
Shetucket at S. Windham, Conn.	406	1913–	12/15/20	8 900	21.9	......	16 000	39.4	......
Quinnebaug at Putnam, Conn...	332	1927–	3/ 6/34	5 200	15.7	......	16 500	49.7	10.3
Quinnebaug at Jewett City, Conn.	712	1918–	11/ 5/27	13 100	18.4	......	29 000	40.7	......

TABLE 3.—(*Continued*)

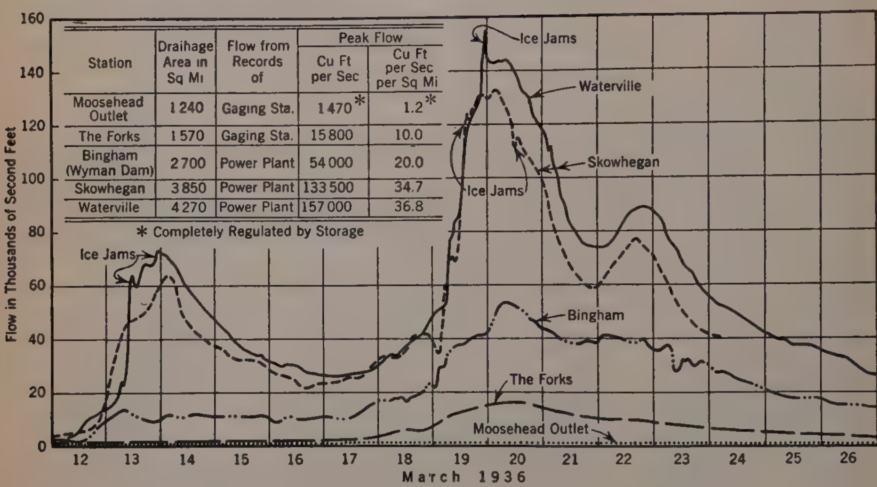
River and Station	Drainage area, in square miles	Period of record	Maximum Discharge Previously Recorded			1936 Peak Discharge			Run-off, in inches, March 12–26, 1936, inclusive
			Date	Discharge: Cubic feet per second	Discharge: Cubic feet per second per square mile	Date March, 1936	Cubic feet per second	Cubic feet per second per square mile	
Connecticut River Basin:									
Connecticut at N. Stratford, N. H.	796	1930–	4/13/33	21 500	27.0	13	28 400	35.7	7.9
Connecticut at S. Newbury, Vt.	2 825	1918–	11/ 5/27	78 000	27.6	19	77 800	27.6	8.5
Connecticut at White R. Junc., Vt.	4 068	1911–	11/ 4/27	131 000	32.2	19	120 000	30.1	8.6
Connecticut at Vernon, Vt.	6 110	....	11/ 4/27	155 000	25.4	19	180 000	29.8	......
Connecticut at Turners Falls, Mass.	7 138	1915–	11/ 4/27	171 000	24.2	19	195 000	27.6	......
Connecticut at Montague Ct., Mass.	7 840	1904–	11/ 5/27	171 000	21.8	19	227 000	28.9	8.8
Connecticut at Holyoke, Mass.	8 390	1880–	11/ 5/27	169 000	20.8	20	222 000	27.4	......
Connecticut at Thompsonville, Conn.	9 637	1928–	11/ 6/27	199 000	20.6	20	282 000	29.3	8.8
Passumpsic at Passumpsic, Vt.	423	1928–	5/19/33	9 540	22.5	18	16 000	37.8	......
White at W. Hartford, Vt.	690	1915–	11/ 4/27	120 000	174.0	18	45 400	65.8	11.8
Ammonoosuc nr. Bath, N. H.	393	....	......	......	......	18	27 400	69.7	......
Ashuelot at Hinsdale, N. H.	420	1907–	3/ 9/20	8 940	21.3	19	16 400	39.0	......
Millers at Erving, Mass.	370	1914–	11/ 4/27	6 350	17.2	19	19 700	53.2	......
Deerfield at Charlemont, Mass.	362	1913–	7/ 8/15	50 600	140.0	18	32 300	89.0	......
Ware at Gibbs Crossing, Mass.	199	1912–	9/17/33	2 960	14.9	19	11 130	55.8	......
Swift at W. Ware, Mass.	186	1910–	4//7/23	2 390	12.8	19	7 580	40.7	......
Quaboag at W. Brimfield, Mass.	151	1909–	3/17/20	1 980	13.1	19	3 760	24.6	8.3
Mid. Br. Westfield at Goss Hts.	52.6	1910–	9/17/33	6 310	120.0	18	8 400	159.8	......
Farmington nr. New Boston, Mass.	92.7	1913–	11/ 3/27	7 900	85.1	18	9 080	97.8	......
Farmington at Tariffville, Conn.	569	1928–	11/21/32	7 610	13.4	......	22 200	39.0	......
Housatonic at Gr. Barrington, Mass.	280	1913–	11/ 5/27	5 690	20.3	19	8 960	32.0	......
Housatonic at Falls Vill., Conn.	644	1912–	11/ 5/27	11 700	18.2	21	14 500	22.5	7.8
Naugatuck nr. Naugatuck, Ct.	247	1918–1924 1928–	......	......	......	......	15 000	60.7	......
Housatonic at Stevenson, Conn.	1 550	1928–	3/ 5/34	23 700	......	......	67 700	43.7	......
Hoosic at Adams, Mass.	46.2	1931–	4/ 1/32	1 030	22.3	18	3 685	79.8	14.5
N. Branch Hoosic at N. Adams, Mass.	39.0	1931–	11/ 4/27	10 218	262.0	18	3 860	98.9	17.6
Lake Champlain Basin:									
Otter Cr. at Middlebury, Vt.	628	1903–1907 1910–1920 1928–	11/ 4/27	13 600	21.7	19	11 000	17.5	......
Winooski at Montpelier, Vt.	420	1909–1923 1928–	11/ 3/27	57 000	135.8	18	20 000	47.7	......
Winooski nr. Essex Junct., Vt.	1 070	1928–	11/ 4/27	116 000	108.2	19	61 000	57.0	......
Dog at Northfield, Vt.	52	1909–1920 1928–	11/ 3/27	8 000	153.8	18	5 700	109.6	11.7
Mad at Mooretown, Vt.	139	1928–	......	......	......	18	9 070	65.3	10.0
Lamoille at Johnson, Vt.	289	1910–1913 1928–1936	11/ 3/27	36 600	126.7	18	10 000	29.9	......
Lamoille nr. Milton, Vt.	692	1929–	4/13/34	16 600	23.0	19	22 000	30.4	......

the way to the mouth. The ice jams were the occasion of much of the damage. The effect of an extensive ice jam in the river above Waterville is indicated on the Waterville hydrograph of March 13.

The peak of this first flood at Waterville, 71 500 cu ft per sec, was not great in terms of the entire water-shed, but from the 1 670 sq miles of intervening drainage area below Bingham, the peak was about 42 000 cu ft per sec, or 37.1 cu ft per sec per sq mile.

In the second flood there were further ice jams, which were caused by up-river ice. These jams can be observed in the Waterville and Skowhegan

hydrographs. One of them broke and passed Waterville at 1:45 P. M., March 19, and reached the Augusta Dam at 4:40 P. M., traveling a distance of 20 miles in 3 hr. The peak of the second flood at Waterville, of 157 000 cu ft per sec, amounted to 52.2 cu ft per sec per sq mile, on the 3 000 sq miles of drainage area between Moosehead and Waterville.



Station	Drainage Area in Sq Mi	Flow from Records of	Peak Flow	
			Cu Ft per Sec	Cu Ft per Sec per Sq Mi
Moosehead Outlet	1 240	Gaging Sta.	1 470*	1.2*
The Forks	1 570	Gaging Sta.	15 800	10.0
Bingham (Wyman Dam)	2 700	Power Plant	54 000	20.0
Skowhegan	3 850	Power Plant	133 500	34.7
Waterville	4 270	Power Plant	157 000	36.8

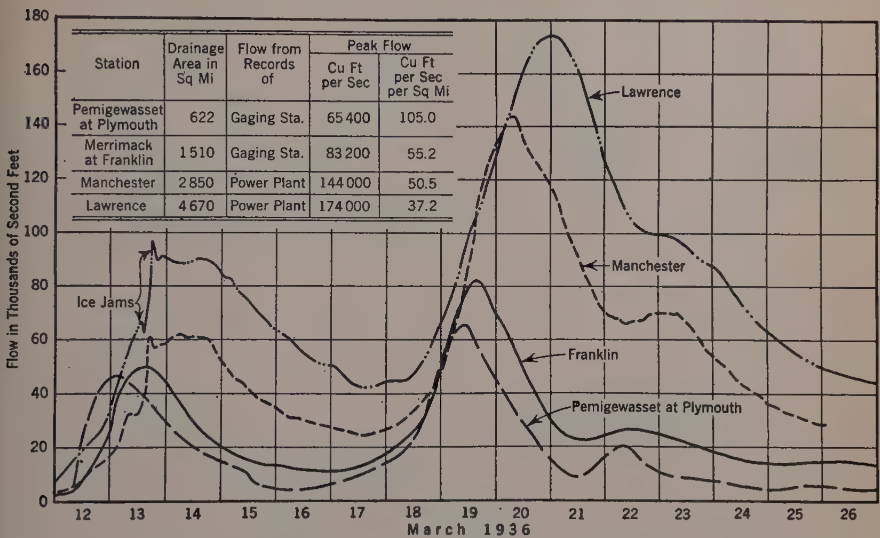
FIG. 7.—KENNEBEC RIVER HYDROGRAPHS, FLOOD OF MARCH, 1936

These hydrographs of the flood on the Kennebec River show graphically the great value of the storage of the Moosehead System. In the period, March 12 to 26, inclusive, 16 300 000 000 cu ft (375 000 acre-ft) was stored in Moosehead, a raised natural lake, and Brassua, an artificial reservoir, and practically no flow was released. The quantity stored is equivalent to 5.7 in. over the Moosehead water-shed of 1 240 sq miles, or 1.7 in (about 23% of the run-off) over the entire drainage area above Waterville. Another measure of the value of this storage is shown by a comparison of the run-off of the Dead River and Moosehead Lake. The Dead River, an important tributary immediately below Moosehead Lake, with no storage and with a drainage area much like that of Moosehead, had a peak run-off of 32.4 cu ft per sec per sq mile, and a total run-off of 8.5 in. from a drainage area of 878 sq miles. On the other hand, there was practically no flow from the Moosehead drainage area, all the run-off going into storage.

Merrimac River.—The flow of the Merrimac River is shown in Fig. 8, together with the flow of the Pemigewasset River, at Plymouth, N. H. At Lawrence, Mass., the shape of the first peak on March 13 and 14 is considerably distorted. An ice jam was formed at a bend in the Merrimac River about a mile above Lawrence in the early evening of March 13, and caused first a drop and then a very rapid rise of the water level.

For the period, March 12 to 26, inclusive, the run-off at Lawrence was equivalent to 9.6 in. over the entire water-shed, which is 49% of the mean annual run-off based on 55 yr of record. At Franklin, N. H., the flood run-off

was 9.8 in., indicating that the run-off was uniformly distributed over the entire water-shed. This fact is also indicated by the high run-off of tributaries all over the water-shed (see Table 3).



Station	Drainage Area in Sq Mi	Flow from Records of	Peak Flow Cu Ft per Sec	Peak Flow Cu Ft per Sec per Sq Mi
Pemigewasset at Plymouth	622	Gaging Sta.	65 400	105.0
Merrimack at Franklin	1 510	Gaging Sta.	83 200	55.2
Manchester	2 850	Power Plant	144 000	50.5
Lawrence	4 670	Power Plant	174 000	37.2

FIG. 8.—MERRIMAC RIVER HYDROGRAPHS, FLOOD OF MARCH, 1936.

The magnitude of the flood peaks on the main river was considerably reduced by valley storage. Between the Lawrence Dam and the junction of the Winnepesaukee and Pemigewasset Rivers, at Franklin, the valley storage amounted to about 17 500 000 000 cu ft (400 000 acre-ft) equivalent to 1.6 in. over the water-shed above Lawrence. The total amount of reservoir storage now developed above Lawrence is 12 000 000 000 cu ft (276 000 acre-ft).

Connecticut River.—The flow of the Connecticut River at five U. S. Geological Survey Stations is shown in Fig 9. Two floods occurred on this river, one having its peak on March 13 and 14, and the other on March 19 and 20. The first peak was confined largely to the middle and lower parts of the water-shed, as indicated by the fact that it amounted to only 11 000 cu ft per sec, or 7.2 cu ft per sec per sq mile, at Gilman, Vt., as compared with 14.7 cu ft per sec per sq mile at Montague City, Mass. This first peak was not as high relatively as the corresponding peak on the Merrimac River. At Montague City the peak of March 13, which reflects some interference by ice jams, was about 115 000 cu ft per sec, and the average daily flow was 110 000 cu ft per sec, or 14 cu ft per sec per sq mile. In the 56 yr of record on the Lower Connecticut River, flows of this magnitude have occurred only in 1895, 1896, and 1927.

On the Connecticut River, the valley storage was relatively greater than on the Merrimac River. Between the Thompsonville Dam (Windsor Locks, Conn.), and Waterford, Vt., the total valley storage was approximately 42 400 000 000 cu ft (974 000 acre-ft), equivalent to 1.9 in. on the water-shed above Thompsonville. The total quantity of water stored in existing power

storage reservoirs in the period, March 12 to 26, inclusive, was about 8 000 000 000 cu ft (184 000 acre-ft) equivalent to about 0.4 in. on the water-shed above Thompsonville.

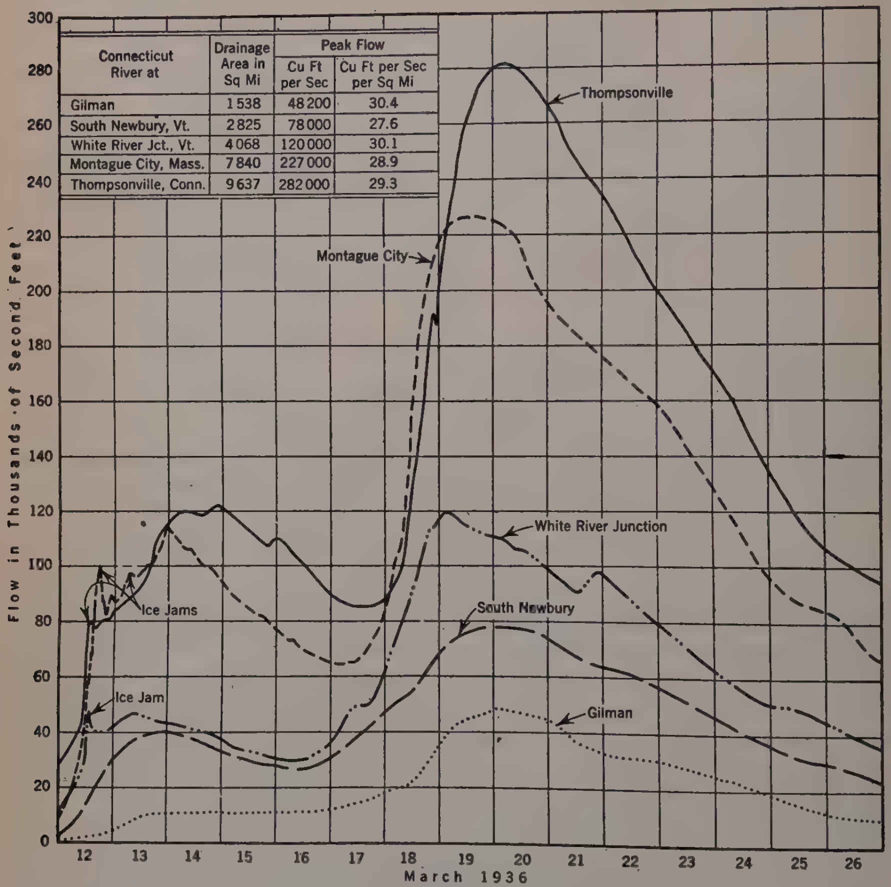


Fig. 9.—Connecticut River Hydrographs, Floods of 1936.

The second peak prevailed over the entire water-shed, the unit run-off being between 27 and 30 cu ft per sec per sq mile for all the stations. For the entire period the run-off at all the stations, except Gilman, was about 8.5 in., and that at Gilman was 7.7 in. This is about 40% of the mean annual run-off (30 yr of record) of 22 in.

Long-Term Records

More long-term records are available in New England than in other regions of the United States. The magnitude of the 1936 flood can be best judged by a review of these records on various streams. Reference is made in this connection to Table 3. The following detailed remarks are also of interest.

Connecticut River.—At Hartford, Conn., a record of river stage has been kept since about 1836. Scattered observations of great flows go back to 1683. On March 20, 1801, there was a great flood, with a river stage of 27.5 ft. From contemporary newspaper accounts, it appears that this exceeded any previous stage within the memory of any one living at that time. The next great flood, which occurred May 1, 1854, reached a stage of 29.8 ft. The flood of November, 1927, crested at 29.0 ft. At the peak of the 1936 flood, the stage was 37.3 ft, greatly exceeding any flood for at least 150 yr. At Holyoke, Mass., records maintained by the Holyoke Water Power Company are available since 1880. The previous maximum flood occurred in November, 1927, when the flow was 169 000 cu ft per sec as against the 1936 flow of 222 000 cu ft per sec.

Merrimac River.—Records have been kept by the water power companies, at Lowell, Mass., and at Lawrence since 1846 and 1848, respectively. The average of the maximum flood each year for the 91-yr record, 1846 to 1936, inclusive, is 46 600 cu ft per sec.

At Lowell, in that period, there have been five great floods: 1852, 1895, 1896, 1927, and 1936. The notes of the late James Bicheno Francis, Past-President and Hon. M., Am. Soc. C. E., the Engineer in charge of the water power at Lowell from 1845 to 1885, state that, in 1785, there was a flood of substantially the same magnitude as in 1852. The latter reached a peak discharge of 108 000 cu ft per sec. In 1895, 1896, and 1927, the flood peaks were 18.9, 20.8, and 17.3 cu ft per sec per sq mile, respectively. In March, 1936, the peak of the "ice flood" was 90 000 cu ft per sec, and that of the week following, 173 000 cu ft per sec. In other words, in 1936, the first flood peak was the fourth greatest in a period of about 200 yr, and the second exceeded by 65% any flood on record.

On the Pemigewasset River (drainage area, 622 sq miles), a tributary of the Merrimac which drains the southern slope of the White Mountains, a record has been kept at Plymouth, N. H., since 1886. The five greatest flood peaks in the 50 yr of record are:

Date	Cubic Feet per Second	Cubic Feet per Second per Square Mile
July 15, 1897	32 800[9]	52.7
April 29, 1923	33 800	54.3
November 4, 1927	60 000	96.4
March 13, 1936	46 200	74.3
March 19, 1936	65 200	105.0

Kennebec River.—At Waterville the approximate height of the great floods since about 1836 has been obtained on a landmark known as the "freshet oak". Elevations of the scars left by the great freshets are as follows:

Date	Elevation, in Feet	Date	Elevation, in Feet
May 22, 1832	104.8	April 30, 1887	102.2
October, 1854	102.8	December 16, 1901	105.14
October, 1869	102.7	March 20, 1936	106.3

[9] Average daily flow. No data available on the flood peak, which presumably was higher.

The flood of May 22, 1832, had an estimated peak of 140 000 cu ft per sec.[10] The flood of December 16, 1901, had a peak flow of 157 000 cu ft per sec, as measured at the Hollingsworth and Whitney Dam about a mile up stream. The flood of March, 1936, also had a peak flow of 157 000 cu ft per sec, as measured at the Hollingsworth and Whitney Dam. Because of the heavy run of ice, the scar left by this freshet was very wide. The elevation given is of the middle of the scar and may not be an accurate record of the maximum flood stage.

Comparison of the 1936 and the 1927 Floods

In spite of the fact that the 1936 flood was greater than any on record on most of the large rivers in New England, the flows per square mile on some of the medium and small water-sheds were greater in the 1927 flood (see Table 3 and Fig. 3).

For drainage areas of about 1 000 sq miles, the maximum flow on record is that of the Winooski River at Essex Junction, Vt., 111 cu ft per sec per sq mile. In 1936, no flow of more than 60 cu ft per sec per sq mile was measured on a drainage area of this size.

For drainage areas of about 500 sq miles, the 1927 unit flows were also much greater. In 1927, the White River (Vermont) had a peak of 174 cu ft per sec per sq mile on a drainage area of 690 sq miles. In the 1936 flood the nearest approach to this unit flow occurred on the Pemigewasset River, at Plymouth (104.8 cu ft per sec per sq mile for a drainage area of 622 sq miles).

As for areas of less than 100 sq miles, in 1927, unit flows of 303 and 257 cu ft per sec per sq mile occurred on drainage areas of 38 and 67 sq miles, respectively—Jail Branch and North Branch, tributaries of the Winooski River. (Detention basins on these two streams were completed in 1935 by the Army Engineers.) No flows of this magnitude were recorded in 1936, the greatest comparable unit flows being 201 and 169 cu ft per sec per sq mile, on two areas of about 30 sq miles each, tributary to the Souhegan River in the central part of the Merrimac water-shed.

Flood Damages

The amount of damage caused by the 1936 flood has been investigated by a number of public agencies, including the Massachusetts State Planning Board, the New Hampshire Flood Reconstruction Council, and the Providence, R. I., and Boston, Mass., District Offices of the Corps of Engineers, U. S. Army. The work of gathering and tabulating these data is largely completed (1937) although some of the figures given are subject to some further correction. These figures do not include estimates of indirect or consequent damages, such as loss of business.

The total flood damage in New England amounted to about $70 000 000, of which $67 893 000 is included in the nine river basins listed in Table 4, and comprises the areas where the damage was heavy. The areas not included

[10] H. R. Doc. No. 658, 71st Cong., 3d Sess.

(the St. John and St. Croix River basins in Northern Maine, and the western slope of Vermont, tributary to Lake Champlain) were not badly damaged.

TABLE 4.—ESTIMATE OF DAMAGE CAUSED BY THE NEW ENGLAND FLOODS OF MARCH, 1936

River basin	Drainage area, in square miles	Damages	
		Total	Per square mile
Penobscot	8 570	$369 000	$430
Kennebec	5 970	1 494 000	250
Androscoggin	3 470	3 574 000	1 030
Saco	1 720	1 401 000	810
Merrimac	5 015	20 148 000	4 010
Blackstone	540	1 879 000	3 470
Thames	1 470	3 633 000	2 470
Connecticut	11 320	34 500 000	3 050
Housatonic	1 930	895 000	460
Total	40 005	67 893 000	.....
Average	......	....	$1 700

In its report of April 25, 1936, the New Hampshire Flood Reconstruction Council estimated the total flood damage in that State at $10 417 924. It was divided as follows:

Highways and bridges		$4 257 214
Other public property (including municipal utilities)	$229 855	
Quasi-public property	39 345	
		269 200
Utilities and railroads	$1 983 813	
Less municipal utilities	49 855	
		1 935 958
Industries: Manufacturing, wholesale, and retail establishments		2 868 857
Recreational facilities		10 000
Farms and rural property		426 695
Other private property		650 000
Total		$10 417 924

The population of New Hampshire in 1930 was 465 293, and the taxable wealth in 1922 was $1 283 000 000. The estimated damage amounted to $21.90 per capita, or 0.81% of the taxable wealth.

The Massachusetts State Planning Board has made an estimate of the total damage in Massachusetts. At the present time (March, 1937) this estimate is about complete, and accounts for damages of $35 773 926, made up as follows:

Highways	$2 943 760
Highway bridges	5 085 343
Water supply and sewerage systems	571 030

Other municipal losses	$754 871
Railroads	1 305 631
Other utilities	1 638 555
Industrial and commercial	16 872 255
Dams	714 650
Residential	5 070 078
Crop lands	817 753
Total	$35 773 926

The population of Massachusetts, in 1930, was 4 249 614, and the taxable wealth, $22 552 000 000. The estimated damage amounted to $6.25 per capita, or 0.115% of the total taxable wealth.

For some of the tributaries estimates of damage are virtually complete. Two are given herein, one of the damage on the Millers River water-shed, the other on the Pemigewasset River water-shed.

The Millers River lies on the eastern slope of the Connecticut River, which it enters a short distance above Greenfield, Mass. The drainage area is 393 sq miles, and the total fall of the main river is about 800 ft in 37 miles. There are eight large towns along the river. Some of the rural land is farmed, but a large proportion is woodland. The towns and industries are centered around the many low-head water-power developments.

There were two flood peaks on the Millers River, the first being accompanied by ice jams which caused much damage. The second peak was considerably higher than the first one, and attained a unit discharge of 53.2 cu ft per sec per sq mile on a drainage area of 370 sq miles at Erving, Mass. This is about three times as great as any other peak observed in the 22-yr record. The damages were estimated, as follows:

Highways	$189 531
Highway bridges	471 200
Water supply and sewerage systems	34 980
Other municipal losses	0
Railroads	226 800
Other utilities	95 000
Industrial and commercial	925 450
Dams	158 500
Residential	241 825
Crop lands	9 060
Total	$2 352 346

The total population of the Millers River water-shed is about 50 000, making the per capita damage about $47.

The Pemigewasset River has a total drainage area of 1 085 sq miles on the southern slope of the White Mountains. The water-shed is largely wooded and is sparsely settled. Most of the permanent population is concentrated in a few towns in the lower valley. There is a large summer population, particu-

larly on Squam and Newfound Lakes and in the vicinity of Franconia Notch. The 1936 flood flows are shown on Fig. 9. The damages were estimated as follows:

Commercial	$57 000
Industrial	225 000
Residential	156 000
Municipal	12 000
Utilities	25 000
Railroads	180 000
Highways	112 000
Highway bridges	30 000
Farms	232 000
Total	$1 029 000

Effect of Ice Jams.—A large part of the total flood damage in 1936, particularly in Maine, was caused by the ice that went out with the peak of March 13. Ordinarily, the spring freshets come after a period of warm weather that to some extent melts and rots the ice sheet in the mill ponds. In 1936, however, the freshet came very early, and the ice went out as hard, "blue" ice. As a result, ice jams accompanied the progress of the flood peak down river.

On the Androscoggin and Kennebec Rivers, the ice jams were particularly spectacular and destructive. On March 20, an ice jam dammed the river at the head of the mill pond of the Gulf Island Hydro-Electric Station. This jam was about ½ mile wide and 1¼ miles long, and extended 20 ft above the water level. When it broke, it created a rise of 1.2 ft in the pond level, equivalent to about 500 000 000 cu ft (11 500 acre-ft), in less than 30 min. An 80-ft highway truss rode the ice jam, and finally passed through one of the Tainter gates on the dam, buckling on the spillway crest as it went through.

On the Kennebec River the greatest damage was due to an ice jam in the tidal part of the river. This jam took out the piers of a modern highway bridge, valued at $260 000 and carried the spans some distance down stream, on top of the ice, in a part of the river where the depth was about 50 ft.

On the Connecticut River, an ice jam at the head of the mill pond above Holyoke, Mass., blocked the river so that it finally cut a new channel to the east of the old one.

Economics of Flood Control

In a period of 200 yr, no other floods in New England have caused damage approaching that of the floods of 1927 and 1936. The question of how much this region is justified in spending on preventive measures presents itself.

In a study of the economics of flood control there are two distinct problems. The first concerns the design and construction of individual river structures that affect the natural flow and the occupancy of the natural flood-plains, and the second concerns the construction of works which will prevent or reduce flood flows on the entire river or in certain localities.

The first problem must be solved by the individual owners of properties of various kinds—dams and power plants, bridges, industrial buildings, river walls, railroad and other embankments, and, of course, homes. Damage to river structures during floods indicates that many of them were constructed without adequate engineering design and supervision.

It is difficult to determine how far it is profitable to go in the design of a structure to prevent possible flood damage where property damage alone is involved. Where life is endangered there can be only one answer, but the temporary loss of the use of property or even its partial destruction once in 25, 50, or 100 yr, may not warrant design and construction to prevent all damage.

Attempts have been made to rationalize the second problem, namely, that of flood prevention or reduction, on a purely economic basis. If definite information were available, the solution would be relatively simple; the maximum expenditure economically justified to prevent recurrence of certain flood damages would be the capitalized present worth of the damages over the period of flood frequency causing them.

Because of the lack of definite scientific data on floods which occurred before about 1850, however, it is difficult to establish an accurate basis for computing the frequency of floods of certain magnitudes. This does not mean that no attempt should be made to make economic studies; it does indicate that all possible information should be collected and studied, impartially analyzed, and tested by all known methods.

Estimating the damages due to the 1927 flood on the Winooski River, in Vermont, to have been \$13 500 000, the flood frequency, 100 yr, and giving consideration to other floods, the U. S. Army Engineers estimated[11] that the expenditure warranted to provide adequate control of this stream would be \$433 000, based on a $3\frac{1}{2}\%$ interest rate.

It is further stated that the Winooski Valley can be adequately protected from flood damage by a series of detention reservoirs designed solely for flood control, but that the cost "is far beyond the amount which can be economically justified."

This conclusion is the one generally arrived at from a study of flood protection on the larger New England streams. For a specific case on a small stream where the damage may run very high, or where the opportunities for flood control are unusually favorable, it may be that flood-control measures can be economically justified.

Prevention of Flood Damages

Most of the practical means of reducing flood damages have to do with such matters as flood warnings, storage reservoirs, detention basins, river channel improvements, or river structures, such as dams, bridges, and levees. Of these items, flood warnings and storage reservoirs are of greatest general interest in New England, since they are the most practical measures. Storage reservoirs may provide flood protection for large areas, although not primarily developed for that purpose.

[11] H. R. Doc. 785, 71st Cong., 3d Sess.

Since the 1927 flood a few detention basins have been built in New England to protect a comparatively small area. Technically, they are perfectly satisfactory, and will serve their purpose if they are maintained properly, but their cost under New England conditions will be found to be prohibitive except in rare instances. The other items mentioned are principally of local interest, but of great importance, and are receiving consideration in many places.

Flood Warnings.—The U. S. Weather Bureau is the formal agency in New England which maintains flood stations that report to it and from which flood warnings are issued if the occasion arises. It is proper to question the adequacy of this service and to make suggestions which may improve it.

The problem may be somewhat different in New England than elsewhere. The distances are relatively short except on the major streams, and on all these streams there are agencies which have a vital interest in the progress of floods, and, therefore, make it their business to keep informed. In some instances, this agency is an utility operating hydro-electric plants over a considerable part of the river. In other instances, as at Lewiston, Me., and Lowell, Mass., the company owning a water power also has an interest in up-stream storage and, for many years, has maintained a private system of communication to keep informed about threatened rises. Under existing conditions such information is of little value to the general public, since it is not given out except as an inquiring reporter may obtain it for his newspaper. Information concerning two important factors affecting floods is not generally available, namely, the extent and water equivalent of the snow cover and the condition of the ground beneath the snow. Numerous observations of precipitation in the form of snow are made by the Weather Bureau and other agencies. Public utilities owning water-power plants, and the winter sports organizations, take scattered observations on the amount of snow cover, but few measurements are made of its water equivalent.

Estimates by trained observers of the amount of snow cover and its water equivalent, at representative points in important areas, are needed. The District Engineers of the U. S. Geological Survey, whose assistants periodically visit the gaging stations in their charge to change charts, would seem to be in an excellent position to take this work on with little additional expense.

This agency also obtains considerable advance information about other conditions conducive to floods, from the reports of its field crews, who make periodic measurements and inspections on all the important streams and many of the tributaries. Again, this information is not made public except on request.

In some years snow falls before the ground freezes and never completely leaves the ground before the spring flood. Such a condition (more likely to be found in Northern New England), is one of the many factors that sometimes combine to cause the extreme run-off, and, consequently, should be a part of the data available to the forecasting agency.

One of two corrective steps is needed. Either a central forecasting agency should be set up to supplement the work of the Weather Bureau, or that body

itself should take on additional duties and correlate all the information collected by private and public agencies, so that it may be made available to the public.

Newspapers and radio commentators invariably exaggerate threatened flood conditions, and the public, unable to judge between facts and exaggeration, usually discounts all such information because of its general unreliability. Regulations providing that none but official information be published or broadcast during a flood crisis might be helpful.

Sometimes, of course, meteorological conditions change so rapidly over small areas, that the event happens before the forecast can possibly reach the affected area. This was the case before the 1927 flood in Vermont.

Storage Reservoirs.—Economically, the most promising method of alleviating flood conditions on New England streams is the construction of storage reservoirs in which the reduction of flood peaks is incidental to their operation for increasing the low-water flow.

Storage reservoirs in New England are normally well down by October, and usually nearly empty by the end of winter, the periods when floods are most likely to happen. The larger reservoirs are not full much more than two months in an average year, and with competent management considerable water can be stored temporarily even on a full reservoir. Any temporary storage reduces the peak flow down stream.

In its report, the Committee on Floods of the Boston Society of Civil Engineers states[12]: "The most desirable policy toward the flood problem here in New England appears to be to encourage in every way the construction of reservoirs for power purposes."

The Advisory Committee of Engineers on Flood Control in the State of Vermont, in a report after the 1927 flood. made substantially the same recommendation, as did the Army Engineers in their reports on flood control on many of the New England rivers.

Effectiveness of Storage Reservoirs for Flood Control.—As a test of the effectiveness of storage reservoirs for flood control, an examination has been made of several existing storage reservoirs to see how much capacity was available at the beginning of various floods of record.

Lake Winnepesaukee (Merrimac Water-Shed).—In Lake Winnepesaukee, 7 000 000 000 cu ft (161 000 acre-ft) of storage is made available by works built about 1845 which permit the lake level to be drawn down 40 in. Restrictions occasioned by the use of the lake in the summer for recreational purposes require that if the gage registers 21 in., or less, the outflow is limited to a daily average of 250 cu ft per sec. Floods begin to cause damage in the Merrimac Valley when the average daily flow at Lawrence reaches about 60 000 cu ft per sec. In the period from 1870 to 1935 there were eight floods of this magnitude, or greater. Table 5 shows the percentage of full reservoir capacity available at the beginning of each. The behavior of Lake Winnepesaukee in the 1936 flood is discused later in detail.

Moosehead Lake (Kennebec Water-Shed).—Damage due to floods in the Lower Kennebec Valley starts with an average daily flow of about 87 000

[12] *Journal*, Boston Soc. of Civ. Engrs., September, 1930, p. 464.

TABLE 5.—STORAGE CAPACITY AVAILABLE IN LAKE WINNEPESAUKEE AT BEGINNING OF LARGE FLOODS

Date	Average daily flow at Lawrence, Mass., in cubic feet per second	Percentage of full capacity available at beginning of flood*
April 21, 1870	89 780	18
March 29, 1877	66 600	77
December 12, 1878	76 300	50
April 16, 1895	65 300	77
March 3, 1896	82 200	11
April 9, 1901	62 500	45
March 4, 1902	61 200	16
November 6, 1927	66 600	71

* Capacity, 7 000 000 000 cu. ft. (161 000 acre-ft.)

cu ft per sec, at Waterville, Me. In the period, 1895 to 1926, five freshets of this magnitude or greater have occurred. In four of these floods, the quantity of water stored in Moosehead Lake was an important factor in reducing flood peaks in the Lower Kennebec River (see Table 6).

TABLE 6.—STORAGE CAPACITY AVAILABLE IN MOOSEHEAD LAKE AT BEGINNING OF LARGE FLOODS

Date	Average daily flow at Waterville, Me., in cubic feet per second	Level of Moosehead Lake		Percentage of total capacity available*
		Date	Elevation, in feet	
April 15, 1895	86 200	April 12	1.8	76
March 2, 1896	111 000	February 28	4.75	37
December 16, 1901	151 000	December 12	1.25	83
June 18, 1917	88 500	June 13	7.55	0
May 1, 1923	134 000	April 30	3.6	52

* Capacity, 20 600 000 000 cu. ft. (473 000 acre-ft.).

After 1926 additional storage was created on the Moosehead Lake drainage area by the construction of Brassua Reservoir, with a capacity of 8 600 000 000 cu ft (197 000 acre-ft). The contributing area is 675 sq miles. On March 12, 1936, only 945 000 000 cu ft (21 700 acre-ft) (5% of its total capacity) was stored in Moosehead Lake, and Brassua Lake was empty. Therefore, at the start of the 1936 flood, practically the entire capacity of the two reservoirs was available for the storage of flood flows.

Effectiveness of Power Storage Reservoirs in Reducing 1936 Flood Peaks. —An examination has been made of the amount by which storage reservoirs reduced flood peaks in the 1936 flood, on several streams where sufficient data were available.

Lake Winnepesaukee (Merrimac Water-Shed).—On March 10, 1936, Lake Winnepesaukee was 22% full. In the period, March 10 to 26, inclusive, the total yield of the Lake Winnepesaukee drainage area was 8 860 000 000 cu ft (203 000 acre-ft) (the equivalent of 10.6 in. of run-off). Of this quantity, 7 500 000 000 cu ft (172 000 acre-ft) was stored in the lake, only a few hundred second-feet being released until March 21, two days after the peak had passed Franklin, N. H.

On Fig. 10 is shown the effect of this storage on the average daily discharge of the Merrimac at Franklin, the point where that stream is formed by the junction of the Winnepesaukee River (drainage area, 480 sq miles) and the Pemigewasset River (drainage area, 1 010 sq miles).

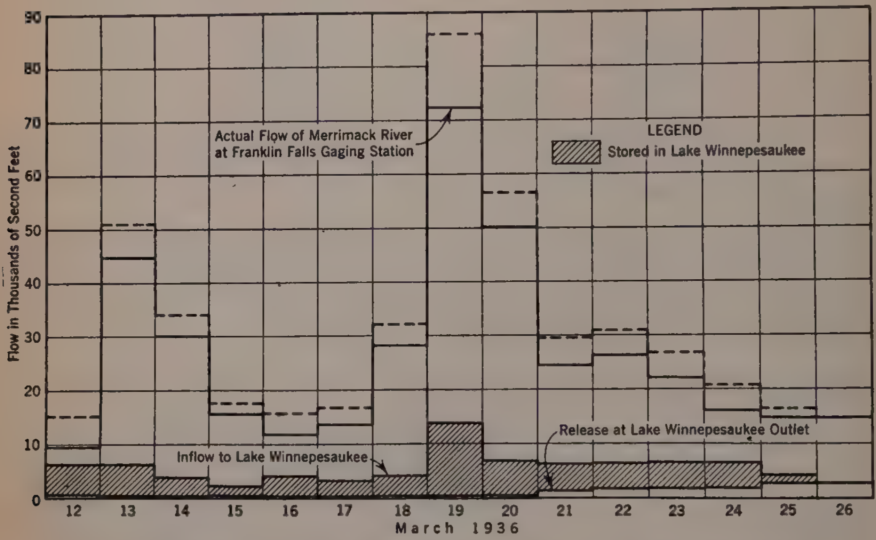


FIG. 10.—EFFECT OF LAKE WINNEPESAUKEE STORAGE, FLOOD OF MARCH, 1936

It seems reasonable to assume that the momentary peak at Franklin, which occurred about 2:00 P. M., on March 19, and amounted to 83 000 cu ft per sec, was reduced by about 9 000 cu ft per sec. The corresponding reduction in gage height is about 3 ft.

At the dams below, the reduction was somewhat less, because of the greater time of transit and the effect of channel storage. It is estimated that at the Sewall Dam and the Garvins Falls Dam, 16 and 28 miles, respectively, below Franklin, the reduction in flow due to Winnepesaukee storage was about 8 000 cu ft per sec, equivalent to a reduction of about 0.75 ft of the flood peak.

Rangeley System (Androscoggin Water-Shed).—During the 1936 flood, 13 000 000 000 cu ft (299 000 acre-ft) was stored in the Rangeley Reservoir System in the period between March 12 and March 26. Releases during the same period totaled only about 420 000 000 cu ft (9 600 acre-ft), and did not begin until March 22, when the flood peaks had passed Rumford, Me. The total quantity stored was equivalent to 5.1 in. of run-off over the water-shed, or to an average flow of 10 000 cu ft per sec for the period.

Because of the number of units comprising the system, it is difficult to estimate the effect of this storage on the flood peaks. However, on the rigorous assumption that the reduction in peak flow was no greater than the reduction in the average flow for the period, it amounted to about 13.5% at Rumford, 84 miles below the lowest reservoir. The corresponding reduction in height over this dam was about 1.5 ft.

Sebago Lake (Presumpscot River).—As an example of complete storage, the behavior of Sebago Lake is interesting. Sebago Lake, a natural lake with an area of 45.6 sq miles, was raised about 8 ft by means of a dam at the outlet, built in 1878. The storage available is about 9 700 000 000 cu ft (223 000 acre-ft) on a drainage area of 360 sq miles. Additional storage in the various ponds tributary to the lake brings the total to about 12 600 000 000 cu ft (289 000 acre-ft), or the equivalent of about a 15-in. run-off from the water-shed. The outlet of Sebago Lake is the Presumpscot River, a stream about 22 miles long which empties into Portland, Me., Harbor.

Between March 12 and March 26, 1936, the lake stored 6 700 000 000 cu ft (154 000 acre-ft), or an 8-in. run-off from the drainage area. In addition, some water was stored in the smaller lakes, but no records of this storage are available. During the same period only 2 300 000 000 cu ft (53 000 acre-ft) was released down the river. On March 22, the day of maximum run-off, an average flow of 2 200 cu ft per sec was discharged, and the equivalent of an average flow of 8 700 cu ft per sec was stored.

In all the storage systems instanced, the storage was obtained by raising natural lakes, and, therefore, the reduction in flood peaks is not so great as would be the case if no lakes had existed in the first place. Also, in the case of the Androscoggin River, the 1936 flood was to a certain degree a "down-river freshet", for, at the head-waters, where the storage is located, the precipitation and temperatures were lower than farther down stream. Nevertheless, the effect of storage was great; to bottle up a run-off of 5 in. until after the peak is past, is of great benefit.

Deerfield River Storage.—The effects of storage were analyzed on one stream, the Deerfield River, where there are no natural lakes. The Deerfield is an important tributary of the Connecticut River, rising in the Green Mountains in Southern Vermont, and flowing south and east to join the Connecticut just below Turners Falls, Mass. The water-shed is mountainous, with steep rocky slopes, and the stream is characterized by fleshy floods and high run-off. Two reservoirs, Somerset and Harriman (formerly known as Davis Bridge), with drainage areas of 30 and 184 sq miles, respectively, have been built by the New England Power Company, the former in 1913, the latter in 1924. Somerset Reservoir completely controls the 30 sq miles above it, the storage capacity amounting to 38.9 in. on the water-shed. The Harriman Reservoir largely controls the 154 sq miles below Somerset, the storage amounting to 14.1 in. on the water-shed.

In the 1936 flood, no water was wasted from Somerset Reservoir. At Harriman Reservoir, except for a few hundred second-feet used for power purposes, no water was released until March 27, a week after the crest of the Connecticut River flood. It is estimated that these two reservoirs reduced the flood peak at Shelburne Falls, Vt., where the drainage area is about 500 sq miles, from 72 000 to 48 000 cu ft per sec, with a corresponding reduction of gage height of 3.3 ft.

Table 7 illustrates further the value of storage reservoirs in time of flood, giving the quantity of water stored and the storage capacity remaining after the 1927 and 1936 floods.

Studies of Potential Developments.—At present (1937) many different plans of flood control by means of storage reservoirs are being developed by various State and Federal agencies. The scope of this paper does not permit a detailed analysis of the effect of any of these projects on flood peaks. Such analyses have been made, however, in several cases, and a few of the conclusions are quoted.

TABLE 7.—BEHAVIOR OF NEW ENGLAND STORAGE RESERVOIRS—FLOODS OF NOVEMBER, 1927, AND MARCH, 1936

River and reservoir	Drainage area, in square miles	Full Reservoir Capacity			Behavior November 3–4, 1927			Behavior March 12–26, 1936		
		In million cubic feet	Inches, depth over drainage area	Corresponding draw-down, in feet	Amount stored, in inches, on drainage area		Percentage capacity remaining, Nov. 5	Amount stored, in inches, on drainage area		Percentage capacity remaining, March 26
					Maximum, per day	Two days		Maximum, per day	Period	
Kennebec River:										
Brassua Lake	675	8 560	30	5.5	‡	‡	‡	‡	3.0	45
Moosehead Lake	945*	20 630	6.5	9.4	‡	‡	‡	‡	5.3	44
Androscoggin River:										
Upper Dam Reservoir	405	8 890	12.2	9.4	0.6	1.0	17	1.1	7.0	26
Middle Dam Reservoir	104*	5 874	17.3	24.3	2.5	2.6	37	2.3	7.2	10
Aziscohos Reservoir	233	9 590	45.0	17.7	1.0	1.1	48	1.8	11.1	38
Umbagoag Reservoir	353*	3 240	9.5	4.0	0.9	1.1	10	0.7	1.8	7
Presumpscot River:										
Sebago Lake	436	9 700	9.7	9.6	‡	‡	‡	0.8	6.6	0
Merrimac River Basin:										
Merrymeeting Lake	12.6	770	16	26.5	‡	0.8	8	‡	9.2	‡
Lake Winnepesaukee	360	7 000	3.7	8.4	‡	1.4	55	1.7	7.0	0
Squam Lake	57.8	2 000	6	14.9	‡	‡	‡	‡	‡	‡
Newfound Lake	96.4	1 300	6	5.7	1.6	3.1	0	‡	‡	‡
Connecticut River Basin:										
First and Second Lakes	83†	3 840†	......	20.0†	1.79	2.36	26.5	1.0	7.0	35
Goose Pond	15	480	20.0	13.8	0.66	1.28	81	‡	8.95	14
Lake Mascoma	126	500	8.5	1.7	0.70	1.12	0	‡	0.77	0
Lake Sunapee	45.1	860	11.5	8.2	‡	‡	‡	2.5	6.1	0
Somerset and Harriman Reservoirs	184†	7 750	....	18.2	3.8	5.2	25	2.4	12.1	19

* Effective. † Combined. ‡ Data not available.

The New Hampshire Water Resources Board has proposed four storage reservoirs on the water-shed of the Merrimac River, to be built primarily to increase the low-water flow of the river and, secondarily, to reduce floods. These reservoirs would have a total capacity of about 18 000 000 000 cu ft (414 000 acre-ft) and a contributing drainage area of 600 sq miles. In a report on the March, 1936, flood[13], Richard S. Holmgren, M. Am. Soc. C. E., Chief Engineer of the New Hampshire Water Resources Board, estimates that the proposed system of reservoirs would have reduced the 1936 peak flow at Manchester, N. H. (drainage area, 2 854 sq miles), from 114 000 to 85 000 cu ft per sec, with a reduction in the height of water at the Amoskeag Dam, at Manchester, N. H., of 3.1 ft. At the Lowell (Mass.) Dam, he estimates that the flow would have been reduced from 173 000 to 145 000 cu ft per sec, with a corresponding reduction in gage height of 3.0 ft.

In 1935–1936, the Army Engineers made a comprehensive report[14] on the Connecticut River on "the efficient development of its water power, the con-

[13] "Extent and Magnitude of the March, 1936, Flood in New Hampshire."

[14] H. R. Doc. 412, 74th Cong., 2d Sess.

trol of floods, and the needs of navigation." In this report they recommend an "initial flood control plan", which would include ten proposed reservoirs operated primarily for the control of floods and, secondarily, for the generation of power. The reservoirs would have a combined contributing drainage area of 951 sq miles, a capacity below the spillway level of 11 700 000 000 cu ft (269 000 acre-ft), and a capacity above the spillway level of 3 670 000 000 cu ft (84 000 acre-ft). The report states (page 85),

"If the reservoirs of the initial program had been in operation during the 1927 flood, they would have been depleted at the time of the flood under normal operating conditions by 50% of their capacity. * * * Six of the reservoirs would have given almost complete protection at the dam site. * * * On the main stream, the peak at White River Junction would have been reduced by approximately 46% from 148 000 second-feet to about 80 000 second-feet."

Detention Reservoirs.—In Vermont, as a result of comprehensive investigations following the 1927 flood, two detention reservoirs were built on tributaries of the Winooski River and put into operation in 1935. The dams were designed by members of the Corps of Engineers, U. S. Army, and built under their supervision.

As originally planned[15], these reservoirs were large enough for power storage as well as flood control As finally built, however, the dams were lower and their operation was restricted to flood control, with uncontrolled outlets of such size that with the reservoirs full the discharge would still remain within the banks of the stream below. Both dams have wide crests, so that their height may easily be increased if it should prove desirable in the future to provide storage for power purposes.

The Wrightsville Dam is on the North Branch of the Winooski River and controls a drainage area of 71 sq miles. The reservoir has a capacity of 895 000 000 cu ft (20 500 acre-ft) to the spillway crest and 1 000 000 000 cu ft (23 000 acre-ft) to the maximum flow line, equivalent to 5.4 and 6.1 in., respectively, on the water-shed.

The East Barre Dam is on the Jail Branch of the Winooski River and controls a drainage area of 39 sq miles. The reservoir has a capacity of 523 000 000 cu ft (12 000 acre-ft) to the spillway crest and 706 000 000 cu ft (16 200 acre-ft) to the maximum flow line, equivalent to 5.3 and 7.8 in., respectively, on the water-shed.

It should be noted that these reservoirs are in the area in which some of the highest unit flows were recorded in the 1927 flood, that at Wrightsville being 257 cu ft per sec per sq mile and that at East Barre, 303 cu ft per sec per sq mile.

In the 1936 flood, the unit flows were not so high on the Vermont streams; nevertheless, the run-off was high enough to constitute a real test of the effectiveness of these detention reservoirs. At Wrightsville, the maximum rate of inflow, computed from the rise in the reservoir, was 3 960 cu ft per sec (55.8 cu ft per sec per sq mile), and the rate of discharge through the outlet at this time was 870 cu ft per sec (12.3 cu ft per sec per sq mile), a reduc-

[15] H. R. Doc. 785, 71st Cong., 3d Sess.

tion in the flood peak of 78 per cent. The maximum rate of outflow was 960 cu ft per sec. The maximum quantity stored was equivalent to 4.2 in. on the drainage area, or 78% of the total capacity of the reservoir. At East Barre, the maximum rate of inflow was 2 035 cu ft per sec (52.2 cu ft per sec per sq mile), and the concurrent rate of discharge was 427 cu ft per sec (10.9 cu ft per sec per sq mile), a reduction in flood peak of 79 per cent. The greatest rate of outflow was 450 cu ft per sec. The maximum quantity stored was 5.6 in. on the water-shed, or about 95% of the capacity to the spillway level.

It is estimated by the Army Engineers that these two detention reservoirs, regulating 110 sq miles, reduced the peak flow of the Winooski River, at Montpelier, Vt., where the drainage area is 418 sq miles, from 23 700 to 19 200 cu ft per sec.

A computation has been made of the potential effect of the Wrightsville Dam on flows similar to those in 1927. A hydrograph was used based in general on the "standard 100-yr flood",[16] with a total run-off of 5.2 in. in 27 hr and a peak flow of about 250 cu ft per sec per sq mile. With the reservoir in operation the peak flow would have been reduced from 17 000 to about 800 cu ft per sec. The maximum storage would have been 89% of the reservoir capacity.

Conclusions

Comparatively little definite information is available about floods, the conditions under which they have occurred, and the inconvenience, loss of life, and economic losses they have caused.

Predictions relative to 1 000-yr and even 100-yr floods are based on meager information. Except for a few records covering less than 100 yr, the basic data are largely legendary. The importance of collecting additional data should be stressed.

The determination of future flood flows at a specific point on any river should include a study of the flood records of all streams having similar physical and climatological aspects.

Existing storage reservoirs have appreciably reduced the flood flows on several streams. Additional reservoir sites, many of which can undoubtedly be economically developed, are available on the head-waters of most of the New England rivers and their tributaries.

In some cases, it will be necessary to provide for joint action by all the water users on a river in order to develop the storage economically. Such action can be furthered by legislation similar to that creating the New Hampshire Water Resources Board. This Board can initiate and carry out comprehensive developments under certain conditions. With regard to some of the larger streams, which pass through several States, it may be necessary to set up interstate compacts before some of the available storage sites can be developed economically.

Except for local improvements, which will undoubtedly be carried out regardless of economic considerations, the possibilities of strictly economic

[16] H R. Doc. 708, 71st Cong., 3d Sess., Figs. 9 and 10.

flood protection on New England rivers other than by storage reservoirs are believed to be scarce. Such possibilities are increased as the concept of the economics of flood protection is broadened by considerations of public policy

Flood-protection works on the upper reaches of the large rivers, and on their tributaries, will have only a very minor effect on the lower reaches, where much of the damage during extreme floods occurs.

Considering the many combinations of conditions which may cause flood flows on the lower reaches of the larger rivers, calculations of stage reductions due to hypothetical reservoirs, or other practical works, on their upper reaches are somewhat academic since they must be based on meager flood-flow information. It seems sufficient to state that any reduction whatever is desirable, but that the extent of such reduction will vary with conditions.

Acknowledgments

This paper could not have been written so soon after the 1936 flood without the co-operation of the public and private agencies that have been collecting data and have made available data which were still being analyzed. Particular mention should be made of the following agencies and individuals: The Boston, Mass., Office of the U. S. Weather Bureau; the District Engineers of the Corps of Engineers, U. S. Army, in Boston, and in Providence, R. I.; the District Engineers of the U. S. Geological Survey in Boston, Augusta, Me., and Hartford, Conn.; the New Hampshire Water Resources Board; the Massachusetts State Planning Board; and, the Engineers of the New England Power Company, New England Public Service Company, Union Water Power Company, and Kennebec Water Power Company. The writer is also indebted to his associate, Mr. Truman H. Safford, whose assistance in preparing this paper was invaluable.

THE NEW YORK FLOODS OF 1935 AND 1936

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Synopsis

Although records, relatively long for this region, indicated a favored location in respect to the occurrence of great storms, within nine months, New York State was visited by two major floods, one in July, 1935, and the other in March, 1936. The flood of July, 1935, was caused by a storm of the thunder-shower type of unusually large aereal extent in the south-central part of the State. Many small streams had peak discharges of more than 2 000 cu ft per sec per sq mile. The damage, especially severe along the small streams, might well be called dynamic damage. The flood of March, 1936, was caused by moderate rains and melting snow which produced record-breaking discharges, both peak and total, on the larger streams, chiefly in the Hudson and Susquehanna River basins. Damage was caused principally by inundation along the larger streams. More and better hydrologic records are needed as part of the basic data for the design and operation of flood preventive and control projects and the solution of the problems of engineering economics involved.

General

It is difficult to speak in general terms of New York State, which, in many ways, is a region of contrasts. In topography, geology, climate, vegetative cover, precipitation, and run-off, this State offers wide variations. It is doubtful whether any other State east of the Rocky Mountains can show more divergent conditions.

In general, Western New York is flat, a rich agricultural region of low run-off in which the Genesee River is the principal stream. The central part of the State is deeply furrowed by glacial action, the most striking evidence of which is seen in the Finger Lakes. Many small streams of this part have cut deep gorges and are characterized by swift, turbulent flow. The principal rivers (Fig. 11) are the Susquehanna, which flows south through Pennsylvania and Maryland to Chesapeake Bay, and the Oswego, which drains the Finger Lakes and empties into Lake Ontario. Eastern New York has two more or less distinct mountain ranges, the Adirondacks and the Catskills. The highest peak in the State is Mount Marcy, in the Adirondacks, 5 433 ft above sea level. This part of the State is drained principally by the Hudson and Delaware Rivers and by numerous smaller streams flowing into the St. Lawrence River and Lake Champlain.

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With the possible exception of the Hudson, New York State has no large interior rivers, principally because it is so divided by major water-shed lines that the run-off goes in all directions. The Hudson, Delaware, and Susquehanna Rivers, all of which rise in New York, flow directly into the Atlantic Ocean. The Genesee, Oswego, and Black Rivers are tributary to Lake Ontario. The northern and northeastern parts of the State are in the St. Lawrence drainage basin, and a small area in the extreme western part is tributary to the Mississippi through the Allegheny and Ohio Rivers.

Fig. 11.—Major Drainage Basins of New York State

The Adirondack region is for the most part heavily forested, the Catskill region much less so. As of 1937, reforestation is being undertaken in a comprehensive manner by the New York State Department of Conservation in nearly all parts of the State, through the purchase and planting of marginal farm land. Unused farm lands are also being reforested to some extent by natural processes. What effect this gradual change in cover will have on stream flow is unknown at present. An investigation by the U. S. Geological Survey, in co-operation with the Division of Lands and Forests of the State Department of Conservation, is under way and eventually should yield valuable information on this subject.

It is well known that the areal distribution of precipitation in New York State is variable, but unfortunately only meager information is available for the areas of maximum precipitation, which are wild and inaccessible. The prevailing winds are westerly or southwesterly, and because the Adirondacks

act as a barrier, the heaviest precipitation in the State occurs in certain areas on the western slopes of these mountains. Although substantiating records are not available, it seems probable that an annual precipitation of more than 75 in. occurs in some parts of this region.

Variations in run-off are noteworthy and can be illustrated by two examples of extremes. During the eight water-years from October, 1924, to September, 1932, the mean annual run-off from the 75 sq miles of drainage area above the gaging station on the East Branch of Fish Creek, near Constableville, was 48.5 in., whereas, during the same period, the run-off from 2 530 sq miles above the gaging station on the Chemung River, at Chemung, was only 13.4 in. All possible gradations of run-off occur between these extreme conditions.

Hydrologic History Prior to 1935

At present, the U. S. Weather Bureau maintains about 175 precipitation stations in New York State, co-operative and otherwise. Precipitation is recorded by other agencies at a few additional points. Relatively few of the continuous records extend back farther than about 1889, at which time the New York State Weather Service was created by Act of Legislature. Three of these continuous records, however, date from prior to 1830. The U. S. Geological Survey maintains at present 102 gaging stations in the State, but the average length of record per station, in 1936, was only about 16 yr. Very few records of stream flow were made prior to 1900. Basic data covering periods relatively so short are inadequate for drawing conclusions as to the effect of the works of Man on stream flow, or for determining whether or not the run-off is characterized by significant trends of change through the years.

A study of available precipitation records discloses the fact that, during the period, 1869 to 1934, few 2-day storms, with 6 in. or more of precipitation, occurred in New York State on areas of any considerable extent. In this connection see Fig. 12, which is compiled chiefly from Part V of the Technical Reports of the Miami Conservancy District (1936 revision). The storm of October 8 and 9, 1903, showed a 2-day precipitation in excess of 6 in. at some stations, and, no doubt, other such storms occurred over small areas, but the interesting fact, whatever its significance, is that during this period of 65 yr, for the last 43 yr of which it appears certain that the records were adequate, the State was singularly free from storms carrying heavy precipitation. Except perhaps the Delaware flood of 1903, the history of New York State shows no floods prior to 1935 comparable with the Potomac and Susquehanna floods of 1889, the Ohio Valley flood of 1913, or the Vermont flood of 1927. In spite of storms since 1934, therefore, there seems to be some evidence that New York State, except possibly in its southwestern part, occupies a favored geographic position with respect to the incidence of destructive floods.

The Flood of July, 1935

During the period, July 6 to 9, 1935, heavy thunderstorms accompanied by torrential precipitation occurred in an area in South-Central New York

extending roughly eastward from near Hornell, in the northern part of Steuben County, to near Oneonta, in Otsego County. Similar disturbances, generally of a less severe nature, occurred in several other isolated parts of Central and Eastern New York, but the storm's greatest intensity was felt in the

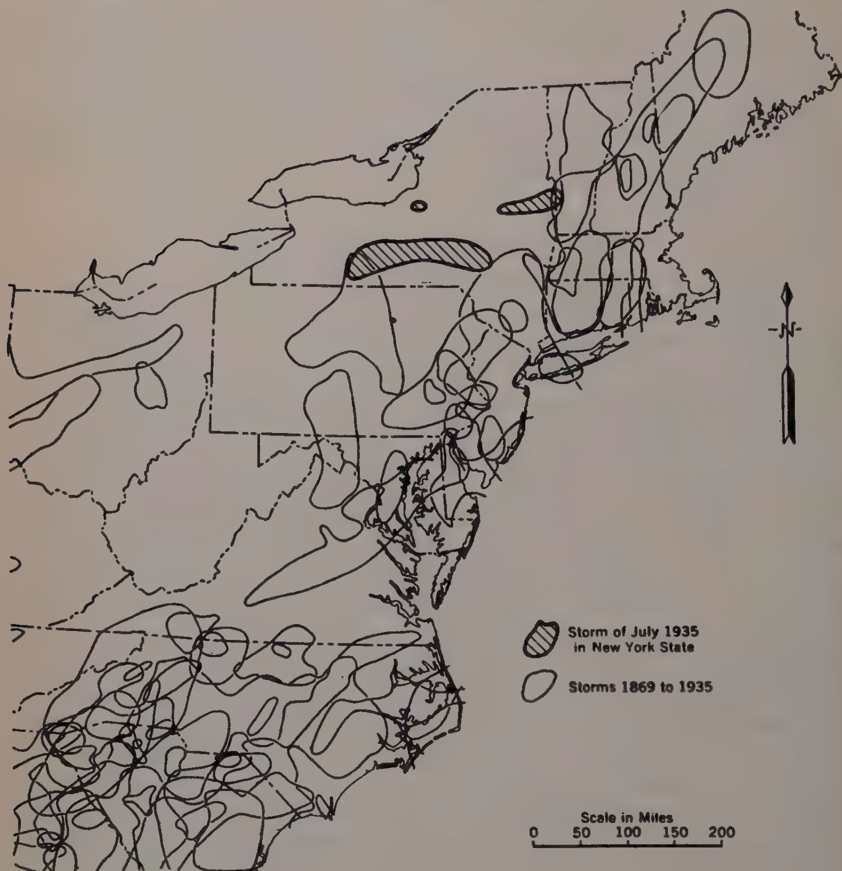

FIG. 12.—LOCATION AND EXTENT OF GREAT 2-DAY STORMS, WITH PRECIPITATION OF 6 INCHES OR MORE

area indicated. The fact that the storm centered where it did seems to have had no particular significance, most authorities being agreed that a similar storm might have occurred under the influence of slightly different atmospheric conditions in almost any other part of the State. In most of the area over which the storm centered, all previous 2-day records of rainfall were broken. The maximum official or semi-official 2-day record for this storm was 10.50 in. at Burdett, in Schuyler County, on July 7 and 8, but unofficial records from other points showed precipitation of 12 to 16 in. occurring in as many hours. Unfortunately, the number of reliable precipitation records in the flood area was altogether too small to give an adequate idea of the condi-

tions, and as the intensity of the storm varied greatly, there is good reason to conclude that greater precipitation occurred at many points then was recorded even unofficially.

One fortunate feature of this storm was that its incidence was such as to cut transversely the main drainage basins of the region in a comparatively narrow zone, with the result that none of the larger streams reached as high a stage as it might have, had the area of the storm been more nearly coincident with its basin. Here, again, there seems to be no reason why the axis of greatest precipitation might not have extended in some other direction than from west to east. Great as the flood damage was, the conditions might conceivably have been such as to produce even more serious results.

The central drainage systems of this region are fed by numerous small streams with generally steep gradients, draining hilly country. Under these conditions and with the extraordinary rainfall, the rate of run-off very quickly approached the intensity of rainfall, and the rise of these streams was very rapid. It was along these small streams that the greatest dynamic damage was done. On the larger streams, losses were caused principally by inundation.

The peak discharges of many small streams with drainage areas of less than 5 sq miles were almost incredibly high (see Fig. 13). There are no

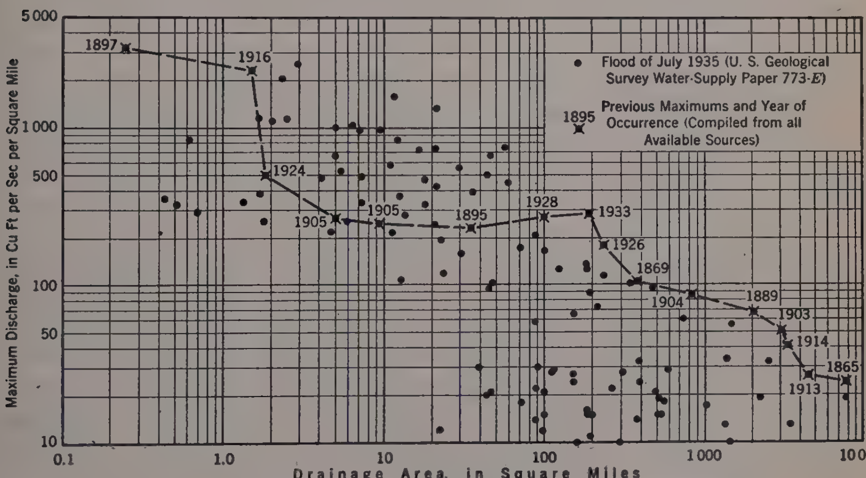

FIG. 13.—MAXIMUM FLOOD DISCHARGES IN NEW YORK STATE.

gaging stations in the flood area on streams of this size, but subsequent determinations by engineers of the U. S. Geological Survey, using the slope-area method, indicate peak discharges of more than 2 000 cu ft per sec per sq mile. It seems probable that some streams draining 1 sq mile, or less, may have had peak discharges of more than 3 000 cu ft per sec per sq mile. The magnitude of the peak flows was much less in the larger drainage areas, although the Chenango River, near Chenango Forks, draining 1 492 sq miles, broke all existing records with a peak discharge of 82 800 cu ft per sec, or 56 cu ft per sec per sq mile.

The outstanding features of this flood, that are of particular interest to engineers, may be summarized as follows:

(1) Large area covered by storm of thunder-storm type.

(2) Unit peak discharges of a magnitude higher than ever before recorded in this part of the United States.

(3) Abnormally heavy and intense precipitation in a hilly country, which caused heavy erosion not only in small stream beds, but even on unbroken hillside surfaces.

(4) Excessive damage in villages situated along small streams, where the encroachment of buildings and other structures restricted the channels and raised the water levels.

(5) Inadequacy of bridge and culvert openings, further complicated by the presence of drift, which resulted in washouts of fills and loss of structures.

(6) Damage by inundation in cities, caused by construction at too low levels, or in unfavorable localities.

A detailed report on the flood of July, 1935, has been published by the U. S. Geological Survey[19].

The Floods of March, 1936

Early in March, 1936, there was considerable snow on the ground throughout most of the State. The Susquehanna River drainage basin above Binghamton had an average of 9.6 in. of snow cover on March 2, with a water content of 2 to 4 in. The Upper Hudson region had as much as 6 in. of water in the form of snow. This general condition was brought about by the fact that temperatures had been below freezing almost continuously since the middle of January, with little run-off except that derived from ground-water.

About March 12, an atmospheric depression moved northward over the Atlantic seaboard, bringing warm weather and general precipitation. This precipitation was not particularly unusual, either in amount or in intensity, but the combination of dense snow on ground frozen to a greater or less depth, high temperature, and rain caused high stages on most of the rivers, except in the northern and western parts of the State. The spring break-up of ice occurred at this time on all except the northern streams. The Tioga and Chemung Rivers, draining areas in Pennsylvania where warm rains melted much snow, reached stages higher than those of the flood of July, 1935. The streams of the Upper Hudson, Mohawk, and Delaware basins were very high. This storm left saturated soils and bank-full streams, and in many sections there was still much saturated snow remaining on the ground.

During the period, March 16 to 19, another atmospheric depression moved slowly northward, bringing from 2 to 3 in. of precipitation. In Central New York this precipitation came as sleet, doing much damage to trees and public utility lines; in the western part of the State it fell as snow. In the southern and eastern areas this rain caused a second flood. Several of the streams in the Mohawk and Upper Hudson drainage basins reached higher

[19] *Water Supply Paper 773-E*, U. S. Geological Survey.

stages than during the preceding week and registered new high records at the gaging stations. The high discharges of this second flood were induced largely by the saturated soils, overlying the frost still remaining in the ground, and the relatively high stages still lingering from the first flood.

The second storm left much snow in the western part of the State; at Buffalo, for example, there was 18 in. on the ground. Warm weather on March 25 and 26, although unaccompanied by rain, caused rapid melting of this snow, with high run-off in the Genesee and other streams. Such stages, although high, were not record-breaking.

It will thus be seen that in the latter part of March, 1936, three separate and distinct floods occurred in New York State. The northern part of the State cannot be considered in the flood area as, in general, no exceptional records were established there. This is also true, although to a less extent, of the western part of the State. The flood stages were most notable in the eastern part of the State and in the Susquehanna and Delaware drainage basins. Moreover, although many new records were established, especially at gaging stations having short-term records, the March floods in New York State were much less outstanding than those occurring in New England, Pennsylvania, Maryland, and other States. The general statement may be made that no New York stream having a drainage area of more than 400 sq miles showed a peak discharge of more than 50 cu ft per sec per sq mile, and few peak discharges of as much as 100 cu ft per sec per sq mile occurred in drainage areas of less than 400 sq miles.

One important and unusual feature of these floods was the occurrence of two storms in close succession, with a resulting long period of high water. High stages existed for more than two weeks, and the total run-off during that time was very large, amounting in some drainage areas to as much as 10 in. over the entire areas. This fact is pertinent in connection with problems of flood control. As the rains extended over a 10-day period, the high stages on the large streams were more notable than on those of smaller drainage areas.

A comprehensive report on the March floods in the northeastern part of the United States is now (March, 1937) in preparation by the U. S. Geological Survey.

Other Floods

Prior to the floods of March, 1936, the flood of March, 1913, was generally regarded as the outstanding spring flood in New York State. It was caused by heavy rains on saturated, snow-covered ground. The entire rainfall occurred in two or three days. Record high water was reached in the Hudson, Mohawk, Susquehanna, and some Northern New York drainage basins. If the rain of March 11 to 21, 1936, had been concentrated in two or three days, or even if the two storms of March 11 and 12 and March 17 and 18 had coincided, the conditions would have been more like those of the flood of 1913, but because of the greater rainfall the results would have been vastly more disastrous.

In November, 1927, rains of high intensity fell on saturated ground in New England and to a less extent in Eastern New York. Serious floods occurred at that time on a few New York streams, notably those rising in New England.

Flood Control

One notable example of the value of effective flood-control measures was brought out during the floods of March, 1936.

In 1913 the Hudson River, at Albany, reached a peak of about 225 000 cu ft per sec, and caused great damage in that city. Later, the Sacandaga Reservoir, with a total capacity of 866 000 acre-ft, was created through the construction by the Hudson River Regulating District of the Conklingville Dam, near the mouth of the Sacandaga River, the principal tributary of the Upper Hudson. This reservoir was designed to control with the reservoir full a Sacandaga River flood with a peak flow of 50 000 cu ft per sec, as a peak of about 35 800 cu ft per sec had been reached in March, 1913. The contributing area is 1 044 sq miles.

At the time of the first flood in March, 1936, the Sacandaga Reservoir was nearly empty. On March 18, the computed peak inflow reached 64 600 cu ft per sec, but the entire flood flow was controlled without difficulty. As a result the peak discharge of the Hudson River, at Albany, was held to about 165 000 cu ft per sec, and a repetition of the great losses suffered in 1913 was prevented.

The inhabitants of the area affected by the flood of July, 1935, had the unique and altogether unpleasant experience of being visited by two disastrous floods within less than nine months. Under these conditions it is but natural that there should arise an insistent popular demand for flood control, particularly in this area and, to some extent, in other parts of the State. The subject is a live one in New York State as well as in neighboring States, but there is little evidence that the general public appreciates what is needed to bring about effective flood control or even effective flood protection.

It is recognized that some relief can be afforded by cleaning and straightening channels, by removing structures encroaching upon the streams, and by increasing bridge and culvert openings within reasonable limits. These measures, however, are merely of an auxiliary protective nature. The backbone of successful flood-control operations must be storage. Whether this storage is provided in one large reservoir, or in many small reservoirs on head-water streams, the total capacity must be relatively large if floods like those of March, 1936, are to be rendered harmless. The general public seemingly does not understand that large storage reservoirs require large natural sites, which are too often non-existent. Should not the public be better informed as to the limitations imposed by Nature on flood-control projects?

Many streams have flood-plains of greater or less extent, which carry off flood waters without difficulty. Most flood damage is caused by the fact that Man has not only occupied these flood-plains with his structures, but has

encroached on the stream beds themselves. Would it not in some areas, particularly in small villages, be cheaper and more desirable to purchase and remove some of these man-made obstructions to flood flows? The opinion is ventured that flood protection involves a problem in engineering economics that may not always have been adequately considered.

The flood of July, 1935, yielded rates of peak discharge to the square mile which before that time had not been considered possible in New York State, and, to this extent, it has provided a new conception of what may be again encountered in any part of the State at any time. It is a puzzling economic problem to determine on what basis designs for flood-control projects should be made. What are the justifiable economic limits?

These floods have emphasized the importance and value of basic data. More and better precipitation stations are needed—stations equipped with automatic recorders, so that rainfall intensities can be studied. Additional gaging stations should also be established, because the best guide for the future is the experience of the past. Long-continued records of precipitation and stream flow at as many strategic points as possible will furnish a basis for better preparation to meet future great floods as they occur. Such floods may be expected, because there is no valid reason why small areas in any part of the State may not experience storm conditions similar to those of July, 1935, or why major parts of the State may not experience conditions such as existed in New England and Pennsylvania during March, 1936.

FLOODS IN THE UPPER OHIO RIVER AND TRIBUTARIES

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Synopsis

The object of this paper is to describe the outstanding characteristics of the unprecedented flood of March, 1936, in the Upper Ohio River and its tributaries, to point out the causes which combined to bring about the occurrence of this flood, and to outline the history of the present flood-control program for these rivers. Recommendations are made for improving the facilities and technique used in forecasting floods in this region.

Flood Factors

Topography.—The Allegheny and Monongahela Rivers converge at "the Point", in the City of Pittsburgh, Pa., to form the Ohio River. The headwaters of the Allegheny are in the Allegheny Mountains, in Pennsylvania and New York State, and those of the Monongahela River are in the Allegheny and Appalachian Mountains, in Northwestern Maryland and Northeastern West Virginia. The Allegheny River flows in a southerly direction, and the Monongahela, in a northerly direction, both skirting the western foothills of these ranges.

According to the Pittsburgh Flood Commission Report of 1912,

"The drainage area of the Allegheny River is 11,580 square miles, and that of the Monongahela River is 7,340 square miles, giving a total of 18,920 square miles. * * *

"The area distribution of the combined basins in percent is as follows:

"New York	10%
Pennsylvania	66%
West Virginia	22%
Maryland	2%
	100%

"The area lying in Pennsylvania is equal to 27.5 per cent of the total area of the State.

"The basins have a greatest length of 290 miles, an average width of about 65 miles and a least width of 46 miles, across the Allegheny at Kittanning.

"The combined area is equal to 9 per cent of the total area of the Ohio Basin, which is 210 000 square miles. * * *

"The principal Allegheny tributaries on the east, 13 in number, have an average length of 47 miles and an average fall per mile of 22.8 feet. The principal tributaries on the west, 10 in number, have an average length of

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37 miles and a fall per mile of 16.7 feet. The Monongahela, on the east has 9 tributaries, with an average length of 45.6 miles and a fall per mile of 31.9 feet; on the west, 7 tributaries, with an average length of 44 miles and a fall per mile of 19.7 feet."

The eastern tributaries of both rivers rise among rough foot-hills at elevations of from 2 300 to 4 800 ft. Those on the west have their source in a high, hilly tableland at elevations of from 1 300 to 2 000 ft.

Although the highest mountains are at the head-waters of the Monongahela, the latter is a mild river even during floods, while in bridge-erection parlance, the Allegheny is a "mad" river. It is, like the Missouri, a treacherous stream; the risk of wash-outs is great during the winter months, and there is no assurance of safety against floods at any time.

It is believed that during the Glacial Period, approximately 75 000 yr ago, the ice flow struck the Allegheny Mountains in Southwestern New York, followed along the Allegheny and Appalachian Mountains, and then followed the Ohio River as far south as Cairo, Ill., depositing enough material in the Eastern Moraine to divert a stream that formerly fed Lake Erie south into what is now the Allegheny. The bluffs of the Upper Allegheny are 700 to 1 000 ft high for many miles, and gradually decrease to a height of 400 to 500 ft at Pittsburgh. The mountains lie within 6 to 10 miles of the stream for many miles at a stretch; their forest cover is gone, the humus is burnt off or washed into the streams, and rocky bluffs are exposed. All these conditions tend to produce rapid run-off.

Rain.—Most of the storms that cause floods in the Upper Ohio basin originate in the Gulf of Mexico. They travel northward up the Mississippi to the vicinity of Cairo, Ill., and thence northeasterly up the Ohio Valley to about Cincinnati, Ohio. From that point they tend to follow one or another of three principal troughs (Fig. 14) which converge at Corry, Pa. Thence, the storms travel in almost an air line to Boston, Mass., and pass into the Atlantic Ocean.

Most of the storms of maximum rainfall travel northeast from Cincinnati. The greatest rain storm of recent years covering a large and continuous area was that of March 23 to 27, 1913, with a rainfall at Dayton, Ohio, of 8.21 in.; Columbus, Ohio, 7.42 in.; Canton, Ohio, 7.39 in.; and Sharon, Pa., 6.56 in. The center of this storm passed 60 miles west of Pittsburgh. On December 14, 1927, a similar storm passed up the Ohio River from Cincinnati to Marietta, Ohio, on the Muskingum River, 297 miles above Cincinnati. Thence, it followed a northerly course through Sharon to Corry, Pa., passing only 40 miles west of Pittsburgh.

The rain storms that bring disastrous floods to Pittsburgh and vicinity are those that continue in an easterly direction from Marietta, swing northeast into the head-waters of the Monongahela River in West Virginia and Northwestern Maryland, and proceed northward along the western foot-hills of the Appalachian and Allegheny Mountains. If the rainfall of March 23 to 27, 1913, or of December 14, 1927, had followed some such path, centering over the Monongahela-Allegheny basin, it would have produced a flood greater than the maximum of record, that of March 18, 1936.

Snow.—In the Allegheny Mountains there is no certainty or regularity in the fall of snow. During the winter of 1907, for example, there was 4 ft of packed snow on the Allegheny Range; in 1909–10 there was 3 ft, which came

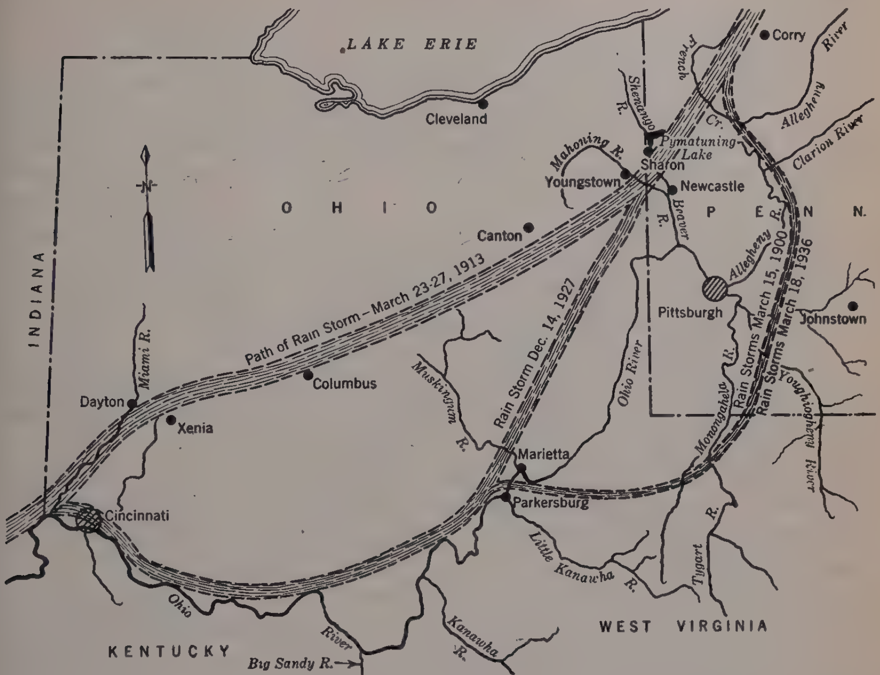


FIG. 14.—PATHS OF HEAVY RAINFALL IN UPPER OHIO BASIN.

generally in very light falls; in none of the years from 1930 through 1935 was there more than 12 in. The winter of 1935–36 was unusually cold and severe. The first snow fell early in December, and was followed by frequent heavy falls throughout the season. As late as February 20, 1936, there was 5 ft of light fluffy snow on the range.

In the last week of February, a thaw occurred, with temperatures of 50° to 59° F in the Upper Allegheny basin and 60° to 66° F in the Monongahela basin. However, the deep snow had drawn the frost out of the ground, and although about 3 ft of snow melted in the low lands, and most of the heavy ice passed out of both rivers, the flood crest of February 28, at Pittsburgh, was only 29.2 ft (flood stage at that point is 25.0 ft).

Again, during the early part of the winter of 1909–10 the bare ground was frozen to an average depth of about 12 in. before the deep snows of 1910 covered the foot-hills and most of the water-shed of the Allegheny River. On the banks of the Clarion River, one of the two largest tributaries of the Allegheny, there was 3 ft of frozen snow. In the early part of March, 1910, the temperature rose, and remained above 40° F continuously for nine days.

Meanwhile, mist and rain fell constantly. A big flood on the Allegheny was predicted. Yet the maximum stage during this thaw and break-up was barely above the flood stage of 25 ft. The blanket of snow had drawn out the frost and had acted as a sponge, holding back the mist, rain, and thawing frozen snow.

Nevertheless, the melting of snow contributes to flood stages, and is always a menace. When there is a blanket of snow, a thaw and a heavy rain storm coming together can produce a flood-stage of more than 30 ft at Pittsburgh. This combination of snow, ice, hard rains, and mild weather was, in fact, the cause of the flood of March 18, 1936, with a stage of 46 ft—the greatest of record at that point.

Ice.—Navigation on the Ohio River as far as Pittsburgh usually continues during the winter months. One of the writers recalls seeing loaded horse-drawn sleds crossing the Ohio on ice from East Liverpool, Ohio, to Newell, W. Va., on December 17, 1904, but such a condition is rare.

One of the severest winters on record was that of 1936. The Allegheny and Monongahela Rivers froze over on December 26, 1935, and by February 21, 1936, there was clear hard ice of a thickness of 20½ in. on the Allegheny and 18½ in. at Lock No. 7, on the Monongahela, 52.6 and 82.3 miles, respectively, from "the Point", in Pittsburgh. It is believed that clear ice froze to a thickness of at least 24 in. in the Upper Allegheny, but no authentic records have been obtained. A similar condition has never been recorded in this section of the United States.

This great body of clear ice with a blanket of deep snow presented a flood menace of unprecedented nature. It was providential that the ice went out of both rivers (except from the head-waters of the Allegheny) on the minor crest of February 28, instead of during the record flood that occurred just nineteen days later.

Forests.—The 1912 report of the Pittsburgh Flood Commission contains descriptions and maps of a large acreage of virgin forest in the Allegheny and Appalachian Mountains. This forest was especially dense in the head-waters of the Allegheny, where there was a fine stand of white pine in eight counties in Northern Pennsylvania and good stands in New York State. In 1937, the only virgin forests left are Federal-and State-owned tracts that include a few parks. The Commonwealth of Pennsylvania has been purchasing forest lands for years, and on October 1, 1934, the State-owned forest land amounted to 1 649 430 acres. In addition, the Game Commission of Pennsylvania has purchased 547 406 acres of forest land for sanctuaries and has turned it over to the possession and protection of the Commonwealth. The Weeks Bill, passed by the Federal Government in 1911, contemplated the purchase of 726 340 acres in Northwestern Pennsylvania to be known as the Allegheny National Forest. By April 26, 1936, forest and surface rights to about 70% of this land had been acquired by the Government.

Notwithstanding the fact that the Federal and State Governments maintain fire towers throughout these areas, and are thoroughly equipped to fight

forest fires, the latter remain the greatest cause of forest destruction and the greatest hindrance to reforestation of unproductive farm lands.

Run-Off.—When the flood of March 15, 1907, came it was freely predicted that it would not be equalled for 100 yr. The engineers of the Pittsburgh Flood Commission, however, did not foresee that the day would soon come when disastrous floods would be caused by the carelessness of Man. They did not realize that the virgin forests would disappear so soon, nor that the time would ever come when hundreds of men would be sent into the upper water-sheds to clear and burn great tracts of fallen, dead trees, to locate and construct dirt highways through the mountains, and all but make parks out of the wilds.

To-day, the virgin forests are all but gone. There are thousands of miles of improved highways and scores of mountain parks; and fishermen, hunters, and careless campers are here, there, and everywhere. The lighted cigarette tossed away by the criminally careless smoker has caused a great increase of timber and land fires that have left in their wake destroyed forests and burnt humus—humus that it will take centuries to replace. Hence, the Man-made floods.

The United States Weather Bureau

Every day of the year other than Sundays and holidays the Pittsburgh Office of the U. S. Weather Bureau publishes a weather map issued by the U. S. Department of Agriculture, giving a forecast for Pittsburgh and vicinity, together with observations taken at 8:00 A. M. It is supplemented by a card giving data on river conditions at various points. About 448 maps and from 86 to 100 river bulletins are mailed daily for local guidance.

The force of the local office has been proficient, accommodating, and of untold value to the public, especially to the river interests and navigators. It is with keen regret, therefore, that it must be recorded that on March 16 and 17, 1936, the local office failed utterly to predict the disastrous flood that was sweeping down both the Allegheny and the Monongahela Rivers, until it was too late to cope even partially with the onrushing tide.

A study of the previous maximum flood of record (1907) and that of 1936 reveals that in both cases the U. S. Weather Bureau either was ignorant of the volume of snow in the mountains at the head-waters of the Allegheny and Monongahela Rivers, or did not appreciate that snow is always a flood menace during March.

The following data are abstracted from the Weather Bureau observations and predictions for three days in March, 1907:[22]

March 13, 1907, Observations at 8:00 A. M.:

Maximum temperature in preceding 24 hr, in degrees Fahrenheit	57
Minimum temperature in preceding 24 hr, in degrees Fahrenheit	49
Rainfall in preceding 24 hr, in inches	1.46
Stage of rivers at Point, in feet	26

[22] In 1907, zero stage was at Elevation 696.8 above mean tide.

March 14, 1907, Observations at 8:00 A. M.:

Maximum temperature in preceding 24 hr, in degrees Fahrenheit..	58
Minimum temperature in preceding 24 hr, in degrees Fahrenheit..	33
Rainfall in preceding 24 hr, in inches	0.68
Prediction made at 3:30 A. M. (of crest stage at Point), in feet...	31
Warning issued at 9:00 P. M. (for crest stage), in feet	34

March 15, 1907, Observations at 8:00 A. M.:

Maximum temperature in preceding 24 hr, in degrees Fahrenheit..	46
Minimum temperature in preceding 24 hr, in degrees Fahrenheit..	31
Rainfall in preceding 24 hr, in inches	0.00
Flood crest, 2:15 A. M. (writers' record), in feet	35.5

Similar data from the observations and predictions for March, 1936,[23] are as follows:

March 16, 1936, Observations at 8:00 A. M.:

Maximum temperature in preceding 24 hr, in degrees Fahrenheit..	45
Minimum temperature in preceding 24 hr, in degrees Fahrenheit..	37
Rainfall in preceding 24 hr, in inches	0.91
Stage of rivers at Point, in feet	19.8

March 17, 1936, Observations at 8:00 A. M.

Maximum temperature in preceding 24 hr, in degrees Fahrenheit..	40
Minimum temperature in preceding 24 hr, in degrees Fahrenheit..	32
Rainfall in preceding 24 hr, in inches	1.68
Stage of rivers at Point, in feet	24.7
Prediction, 8:00 A. M., for forenoon of March 18, in feet	32 to 33

March 18, 1936, Observations at 8:00 A. M.:

Maximum temperature in preceding 24 hr, in degrees Fahrenheit..	34
Minimum temperature in preceding 24 hr, in degrees Fahrenheit..	30
Rainfall in preceding 24 hr, in inches	0.72
Stage of rivers at Point, in feet	42
Prediction (made at 8:00 A. M.) of crest stage, in feet	43.5
Crest reached at 9:00 P. M., in feet	46

Something important was lacking in the Weather Bureau in 1907. It was still lacking in 1936, when the prediction of March 17 was 10 ft in error, and that of the following morning still under-estimated the crest by 2.5 ft. Why these errors?

During the week preceding the flood of March 15, 1907, one of the writers crossed the Tussey Mountains, 16 miles east of Tyrone, Pa., in two places, at elevations of 2 100 and 2 200 ft, respectively. There was about 12 in. of packed snow at the foot of the mountains and 48 in. of light fluffy snow on the summit, equivalent, say, to 4.8 in. of rain. The run-off at that time and place was 70 to 80 per cent.

[23] In 1936, zero stage was at Elevation 694.0 above mean tide.

Immediately before the thaw of February 24, 1936, which resulted in the flood of February 28, of 29.2 ft, the personal records of one of the writers show that there was 60 in. of light fluffy snow on the crest of the Tussey Mountains. As mentioned previously, it happened that no floods came from the 3 ft of melted snow. However, there was 3 ft of packed snow at Cresson, Pa., at the head-waters of the Conemaugh River in the Allegheny River basin, and about 5 ft on the top of the Allegheny Mountains, which did contribute to the flood of March 18.

The thirty-five local Weather Stations in the Allegheny-Monongahela basin reported the snowfall in their daily reports, but gave only meager data regarding the amount of snow on the ground. This deficiency in the record is in part due to the fact that the majority of the stations are in cities and towns instead of in rural districts. However, the primary reason for the deficiency is that the pay of recorders is rediculously small. The pay per month for reporting the weather, daily, is $10. For rainfall reports the observers are paid at the rate of 50 cts per in. of rain.

The Pittsburgh Office of the Weather Bureau claims that it failed to receive many reports from stations along the Allegheny and Monongahela Rivers on account of broken communications by telephone and wire. It is also stated that those stations are manned by agents having other jobs and that the $10 is so much "extra money." There are twelve such weather stations and six rainfall observation stations in the Allegheny basin. The U. S. Army Engineers report the weather and rain conditions to the Weather Bureau from the locks and dams on the Allegheny and Monongahela Rivers and down the Ohio as far as Wheeling, W. Va., without cost.

The local U. S. Weather Bureau Office erred in its predictions for March 15, 1907, and March 18, 1936, for the one and simple reason that it failed to receive adequate weather and rain reports.

The time has long since come when the Pittsburgh Metropolitan Area should be served by adequate weather stations along the Allegheny and Monongahela Rivers and their main tributaries, each fully equipped with modern appliances and manned by a competent employee on a yearly salary. Properly equipped stations should also be established in the mountains, at 50-mile intervals from the Southern Appalachians to the Northern Alleghenies. The observer at each mountain station should be a young, healthy man, preferably a graduate forester.

All stations, both along the rivers and in the mountains, should have private broadcasting outfits, and the mountain stations should broadcast data at stated periods so the public could also listen in. If the local U. S. Weather Bureau Office had known the actual conditions in the mountains on March 16, 1936, and had had the benefit of frequent, accurate, radio reports, forecasts and warnings could have been issued to the public in ample time to have prevented at least $100 000 000 of the loss that occurred in Pittsburgh and vicinity on March 18. A 24-hr notice of the flood would have saved one single department store, for example, from a direct loss of between $750 000 and $1 000 000.

Three great navigable rivers flow through Pittsburgh, the tonnage on one of them, the Monongahela, being greater than that on any other inland river in the United States.

The railroad tonnage over bills of lading in Pittsburgh is the greatest of any city in the country.

Three Federal trunk-line highways, and many highways of less importance, pass through the community.

The direct recorded loss from the flood of March 18, 1936, in the City of Pittsburgh was equal to the total flood loss in 1936 in any one State except Pennsylvania.

Notwithstanding all these facts, the facilities furnished the Pittsburgh Office of the U. S. Weather Bureau for obtaining intelligent reports from the Allegheny and Monongahela Rivers are pitifully meager. This situation is a disgrace. Pittsburgh's most disastrous flood is yet to come.

If the local Weather Bureau continues to be "starved" financially, and is not equipped by intelligent salaried men for reporting weather and rain conditions throughout the 19 000 sq miles of the contributing water-shed, then the entire Weather Bureau should be taken out of the Department of Agriculture and become a department under the jurisdiction of the Chief of Engineers, U. S. Army.

Pittsburgh Floods

There have been eighty-six floods in Pittsburgh since 1856—somewhat more than one per year. According to estimates compiled by the writers the local flood losses during the three decades prior to 1936 have averaged about \$2 000 000 annually. The two floods most disastrous to the Pittsburgh District have already been indicated as those of 1907 and 1936.

Flood of 1907.—At its crest of 35.5 ft,[24] the flood of March 15, 1907, inundated 52% of the "Golden Triangle," the 218-acre central business section. The direct loss to Pittsburgh and its immediate vicinity amounted to \$6 000 000, and the property and indirect loss was placed at \$50 000 000. The supply of water, heat, and light, and most of the local transportation facilities were maintained, although the steam railroads were paralyzed for three days.

Prior to this flood there was considerable snow on the head-waters of the Allegheny and Monongahela Rivers, but on account of the low temperature its thawing was not the main contributing factor. Between March 12 and March 15, rainfall was general over the entire tributary basin. Precipitation varied from 0.20 in. to 3.25 in. on the Allegheny water-shed and from 1.50 in. to 4.25 in. on the Monongahela water-shed. The crests of the two streams reached the Point 2 hr and 15 min. apart. Had they coincided, it is estimated that they would have produced a flood stage of 40 ft[24] at Pittsburgh.

Flood of 1936.—The flood of 1936 reached its crest during the night of March 17–18, at which time 62.4% of the Golden Triangle was under water. The basements of all the office buildings below Smithfield Street and Upper Grant Street were flooded. All power, elevator, and electric service was

[24] Based on zero stage at Elevation 696.8 above mean tide.

shut down. No one was allowed to pass the Federal guard lines into the "Golden Triangle" without a permit from the Director of Public Safety. The business of the city was paralyzed for ten days. Fortunately, so thorough were the regulations of the State Board of Health and the City Department of Health that no epidemic followed the flood in the city and vicinity, and no appreciable increase occurred in the normal death rate.

No precise estimate is available covering both the direct and indirect damage to the Pittsburgh District during this flood. A preliminary estimate by the Pittsburgh Chamber of Commerce placed the damage at about $200 000 000. No estimates have been made of the losses that occurred between the sources of the Allegheny and Monongahela Rivers and the Point, although hardly a town escaped great damage. The disaster also continued down the Ohio River 968 miles to Cairo Ill.

It is impossible to describe the distress suffered on that memorable day of March 18, 1936, especially among the poorer classes, who lost their meager all along a river-bank frontal of more than 1 000 miles. One illustration, out of thousands that could be cited, must suffice. Sharpsburg, Pa., is a little suburb of Pittsburgh, on the right bank of the Allegheny, 6.7 miles above the Point. When the flood waters had receded sufficiently to start shoveling out the mud from hundreds of homes of the middle and poorer classes so many pianos and pieces of furniture and other household goods were dumped on the sidewalks and streets to be hauled away and destroyed that transportation on the highways was blocked.

As to the causes of the flood: The severe winter and heavy snowfall of 1935–36 have already been mentioned. After the minor flood of February 28, the rivers dropped. From March 1 to March 15 mild weather prevailed; the average temperature was 39° and the total rainfall during the period was 1.28 in. The average river stage was 19 ft. All signs pointed to a mild break-up of winter, with the thermometer reaching 61° on March 15 and the river gage at the Point recording 20.8 ft.

General rains began the next day, however. Precipitation on March 17 and 18 varied from 1.33 in. to 3.76 in. on the Allegheny water-shed and from 2.17 in. to 3.79 in. on the Monongahela. The crest of the Monongahela flood reached the Point at 6:00 P. M. of March 17, and that of the Allegheny arrived three hours later, the Monongahela meanwhile remaining stationary.

The Allegheny River flows faster than the Monongahela, and the fact that the crest of the latter generally reaches the Point first can be accounted for by the shorter length of the stream and by the fact that storm centers usually pass over the head-waters of the Monongahela several hours before reaching the drainage basins of important feeders of the Allegheny.

It is a striking coincidence that the Kiskiminetas River, emptying into the Allegheny River 40 miles north of Pittsburgh, and the Youghiogheny River, joining the Monongahela River 15 miles south of that city, should have supplied the major part of the flow at the crest of both the 1907 and 1936 floods. The rapid travel of the flood crest is especially pronounced in the Youghiogheny, which is a rapidly flowing stream from source to mouth.

The Possibility of Greater Floods

The disastrous flood of 1936, has opened deaf ears, and has quickened the conception of the importance of adequate flood control. The public has come to realize that actual damages were only a fraction of the loss that might have occurred. It may seem absurd to suggest that Pittsburgh is a lucky city, but it is a fact.

Many of the conditions that tend to produce a disastrous flood are present in the drainage basin of the Allegheny and Monongahela Rivers. Among these conditions may be mentioned: (1) Steep summit approaches of the Allegheny and Appalachian Mountains; (2) high percentage of run-off; (3) rapid fall of stream beds; (4) ground frozen 1 ft to 3 ft before deep snow falls; (5) severe winters; (6) deep snows every four to six winters; (7) thick ice every five to eight winters; (8) warm spells either the latter part of February or early in March; and (9) hard rains coming about March 15 to 20.

The combined incidence of these conditions was illustrated in the flood of 1936, but not to the fullest possible extent. For example, according to estimates by the writers:

1.—If the rain of March 17–18, 1936, had not turned into snow above Oil City, Pa., the flood crest stage at the Point on March 18 would have been at least 48 ft.

2.—If the great rainfall of March 23 to 27, 1913, had centered over the drainage basin of the Monongahela and Allegheny Rivers instead of about 60 miles west of the basin, the flood height at the Point would have reached 48 ft.

3.—On December 14, 1927, there was a flood of 30.2 ft at the Point. If the rain storm on that date had passed over the drainage area of the Monongahela and Allegheny Rivers, instead of 40 miles west of the Point, the crest would have reached 45 ft.

4.—In 1936, if the thick ice that came out in the break-up of February 16 and 17 had combined with the floods of February 28 and March 18, the crest would have reached 54 ft.

It is of interest to examine one such possibility in detail. If the floods of February 28 and March 18 alone had combined, there would have been a flood of 54 ft at the Point, with enough ice left in the head-waters to form heavy gorges. The ice in the Allegheny River might first have swept the Highland Park Bridge off its foundations, and then the Sharpsburg Bridge. Next, it might have gorged on Herrs Island, and after doing great damage there, might have broken and moved down through the 16th Street and Pennsylvania Railroad Bridges, only to lift the 9th Street Suspension Bridge off its piers, causing it to turn over and hold the ice gorge like an umbrella in a wind storm.

Before the anchorages of the bridge were stressed to the breaking point, the ice might have been packed solid to the bed of the river and piled up from 15 to 20 ft above the flood height at that time. It would soon have

extended to the concrete wall of the elevated section of the Pennsylvania Railroad on the North Side, and by that time the flood in the Golden Triangle would have reached the appalling height of 55 to 60 ft, and the Allegheny River would have been discharging its flood water through the center of the city.

What damage the thick clear ice, rushing through the main streets, would have done to the old buildings and even to the modern sky-scrapers is difficult to state. Its impact against the walls at a velocity of from 8 to 12 ft per sec would certainly have wrought untold damage and destruction. It is certain that most of the old buildings would have quickly collapsed, and this collapse, in turn, would have menaced the stability of the modern buildings. The loss of life and destruction of property would have been appalling. This is no fantastic imagination, but a conclusion which is the product of many years of technical experience in the field. Pittsburgh, the "Lucky City," may some day have such a combination of floods. The following tabulation shows the estimated maximum discharge at Pittsburgh that would have resulted from a combination of the floods of February 28 and March 18, 1936. It is based on gage readings taken March 18, on the tributaries of the Allegheny and Monongahela Rivers:

Estimated Maximum Discharge at Pittsburgh, from the:	Cubic Feet per Second
Allegheny River	473 000
Monongahela River	322 000
Total at Pittsburgh	795 000

This discharge represents a gage height of 54 ft at the Point, according to the extended revised rating curve for Pittsburgh of June, 1936. It is equivalent to a run-off rate of 41.5 cu ft per sec per sq mile, which is 25% in excess of the tabular value published by the Commonwealth of Pennsylvania for a drainage area equal to that of Pittsburgh. The actual run-off rate on March 18, at Pittsburgh, was 97% of the tabular value. The run-off rate on the Susquehanna River, at Harrisburg, Pa., during the same flood, was 21% in excess of the tabular value.

Proposed Plans for Controlling Floods

Although it is not the intention of the writers to enter upon an elaborate description and analysis of the various groups of retarding reservoirs that have been proposed for flood protection on the Allegheny-Monongahela River System, a brief discussion of the nature and action of these reservoirs is pertinent.

The history of floods in the Pittsburgh District begins in 1754, when George Washington, then a young man, nearly lost his life while crossing the flooded Allegheny River on an improvised raft. The site of this occurrence is now marked by the "Washington Crossing Bridge" at 40th Street, in Pittsburgh. Since then, the problem of controlling floods on the Allegheny

and Monongahela Rivers has been discussed on many occasions, but it was not until early in the Twentieth Century that the population and wealth in these river valleys increased sufficiently to justify serious consideration of a comprehensive program of flood control by reservoirs. The flood of March, 1907, rising to a stage of 38.5 ft (based on present pool level) at Pittsburgh, furnished the impetus for the creation of the Flood Commission of Pittsburgh.

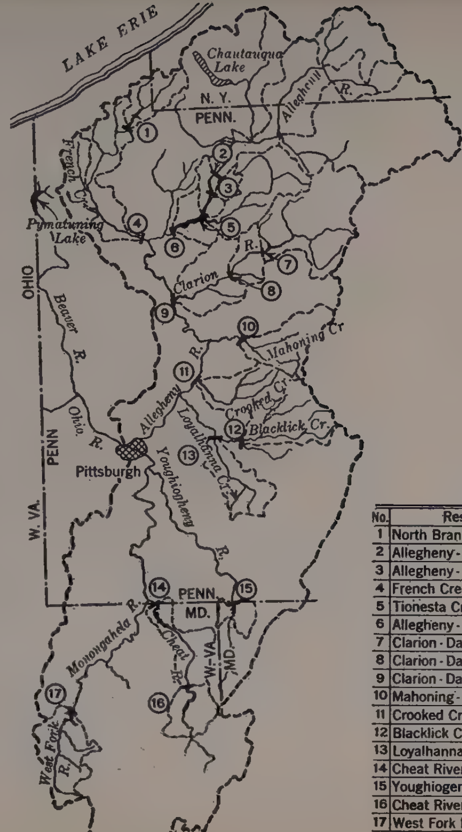
This was essentially a citizen's committee, organized under the auspices of the Chamber of Commerce. The Engineering Committee of the Flood Commission, at the time of the publication of its voluminous report of 1912, was as follows: E. K. Morse, M. Am. Soc. C. E., *Chairman;* the late Emil Swensson, M. Am. Soc. C. E.; the late William Glyde Wilkins, M. Am. Soc. C. E.; George S. Davison, Past-President, Am. Soc. C. E.; the late Paul Didier, M. Am. Soc. C. E.; the late Julian Kennedy; the late Morris Knowles, M. Am. Soc. C. E.; and George M. Lehman, M. Am. Soc. C. E., Engineer in Charge. The Principal Assistant Engineer was Kenneth C. Grant, M. Am. Soc. C. E.

Funds for the work of the Flood Commission were obtained by asking the owners of property affected by floods to contribute at the rate of 1 mill per dollar of assessed valuation. Substantial contributions were made by the City of Pittsburgh and the County of Allegheny. A total of approximately $124 000 was expended from 1908 to 1911 in carrying on extensive surveys and investigations, and a publicity campaign.

The first engineering investigations of the Commission were predicated on the assumption that flood protection would be secured primarily by the construction of river walls. Thorough surveys were made of the river channels within the city limits of Pittsburgh, and hydraulic and economic studies were made to determine the feasibility and cost of such walls, and to establish their most desirable location. It developed from these studies that serious difficulties and uncertainties would be encountered in designing a wall which would provide flood protection without the assistance of reservoirs or channel improvement.

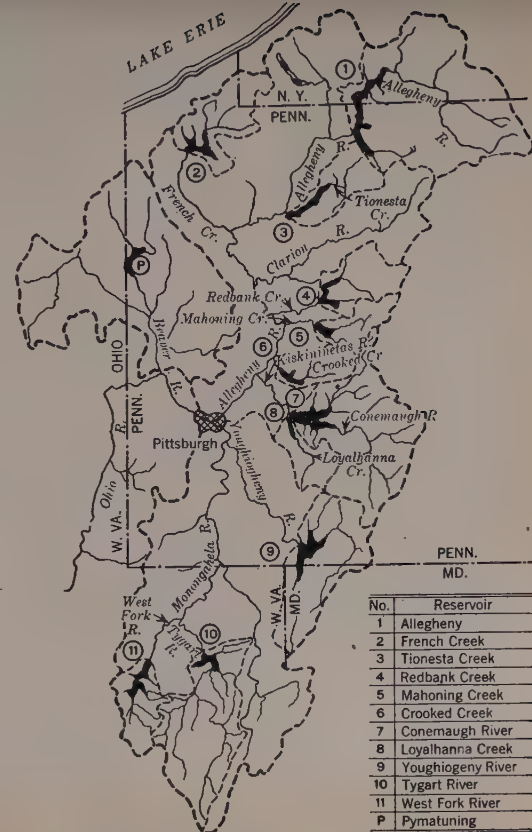
As the work progressed, the possibility and advantage of securing flood control by reservoirs on the head-waters and tributaries of the rivers became more and more apparent, and later investigations included extensive surveys of numerous reservoir sites scattered over the drainage area of about 19 000 sq miles above Pittsburgh. The selection of these sites was based on a reconnaissance including every tributary with a drainage area of more than 50 sq miles. The studies included detailed analyses of costs, and of the flood-lowering effect of various groups of reservoirs. They also developed the possibility of using part of the reservoir capacity to regulate the dry-weather flow of the rivers in the interest of navigation, sanitation, hardness control, and acidity control.

The ingenuity and thoroughness of the hydraulic studies can be admired even after the lapse of a quarter century that has seen great advances in the technique of flood control by reservoirs.



No.	Reservoir
1	North Branch French Creek
2	Allegheny - Dam No. 3
3	Allegheny - Dam No. 2
4	French Creek
5	Tionesta Creek
6	Allegheny - Dam No. 1
7	Clarion - Dam No. 4
8	Clarion - Dam No. 3
9	Clarion - Dam No. 1
10	Mahoning - Dam No. 2
11	Crooked Creek
12	Blacklick Creek
13	Loyalhanna Creek
14	Cheat River - Dam No. 1
15	Youghiogeny River - Dam No. 2
16	Cheat River - Dam No. 2
17	West Fork River

FIG. 15.—SEVENTEEN-RESERVOIR FLOOD-CONTROL PROJECT, PROPOSED BY FLOOD COMMISSION OF PITTSBURGH, PA., IN 1912.



No.	Reservoir
1	Allegheny
2	French Creek
3	Tionesta Creek
4	Redbank Creek
5	Mahoning Creek
6	Crooked Creek
7	Conemaugh River
8	Loyalhanna Creek
9	Youghiogeny River
10	Tygart River
11	West Fork River
P	Pymatuning

FIG. 16.—ELEVEN-RESERVOIR SYSTEM PROPOSED BY CORPS OF ENGINEERS, U. S. ARMY, IN 1928

The Flood Commission secured the establishment of numerous stream-gaging stations and rainfall gages. It collected a great mass of information on matters directly and indirectly related to flood control, such as stream flow, rainfall, topography and geology, forestation and agricultural development, location of coal, gas, and oil fields, railroads and other transportation lines, flood damages, and industrial developments. This information, together with the engineering studies and conclusions of the Commission, was set forth in the report of 1912. The final recommendations of the Commission proposed that seventeen reservoirs be constructed on the head-waters or tributaries of the Allegheny and Monongahela Rivers, for the primary flood protection of all communities and industries on the banks of these rivers. The location of these reservoirs is shown in Fig. 15. Flood walls were also recommended as a secondary line of defense.

In order to convey some idea of the merits of the seventeen-reservoir system of the Flood Commission, in comparison with reservoir systems subsequently proposed, it is desirable to review the general topographical and industrial conditions governing the selection of reservoir sites in Western Pennsylvania.

Although the sources of the Allegheny and Monongahela Rivers and many of their tributaries are in the high mountains that border the eastern edge of the combined drainage basin, the major reaches of the main rivers and their larger tributaries are located in the Allegheny Plateau. The even and horizontal skyline of this plateau shows it to be a plain which has been elevated in comparatively recent geologic times. Since this elevation, the rivers and streams of the region have carved a fully developed drainage system of deep and narrow valleys. In the Pittsburgh region, where the characteristic topography is clearly developed the main valleys have a depth of 400 to 500 ft below the hill tops.

The rocks composing the plateau consist, in general, of the soft sandstones, shales, and coal seams of the Pennsylvanian period, and are bedded nearly horizontally, with a gentle dip to the southwest. This dip causes the main river channels to lie to the west of the median line of their drainage basin, so that the tributaries entering from the west are shorter and less numerous than those entering from the east. In general, the rock structure of the Allegheny Plateau is remarkable for the absence of hard and erosion-resistant strata. The lack of such strata causes the gradients of the larger streams to be uniform over long distances, to the joy of the railroad engineer and the despair of the water-power engineer. The absence of waterfalls, canyons, or gorges on the main rivers of such a rugged region is unusual and striking. Except in its extreme northwestern part, the region is not glaciated, and, in general, rock strata are exposed in the river beds or have only a shallow cover. (The latter statement does not apply to the glaciated region or to the main channel of the Allegheny River, which contains an outwash train of glacial gravel.) Because of these topographic characteristics, one might say that on a typical river or stream of this region every cross-section is a

potential dam site. In so far as natural conditions are concerned, no site has outstanding advantages over numerous other sites on the same stream. In this district the engineer cannot hope to find a canyon dam site lying below a broad flat valley. If topography were the only factor to be considered, it would be correct to state that storage in this region is very easy to obtain, but expensive per acre-foot.

The great obstacle to the construction of retarding reservoirs on the rivers of Western Pennsylvania is furnished not by Nature but by the works of Man; namely, railroads and industrial developments. Several great trunk railroad lines and many branch railroads follow the easy and uniform grades of the river valleys. In the lower and more densely settled parts of the main valleys, it is almost impossible to displace or relocate the railroads or to build a dam of any height without interfering with important industries

TABLE 8.—DATA ON RESERVOIRS IN SEVENTEEN-RESERVOIR PROJECT PROPOSED BY THE FLOOD COMMISSION OF PITTSBURGH IN 1912

Reservoirs*	Controlled drainage area, in square miles	STORAGE CAPACITY In acre-feet		STORAGE CAPACITY In inches of run-off	Percentage of main basin controlled
Allegheny Basin:					
Loyalhanna	277	94 500		6.4	2.4
Black Lick	414	33 400		1.51	3.6
Crooked	287	75 000		4.9	2.4
Mahoning No. 2	335	54 500		3.05	2.9
			257 400	....	....
Clarion No. 1	1 212	116 000		4.06	10.5
Clarion No. 3		112 000			
Clarion No. 4		35 300			
			263 300	....	....
French (exclusive of North Branch)	1 009	76 500		1.42	8.6
North Branch of French	217	49 000		4.24	1.9
Tionesta	477	83 500		3.29	4.1
			209 000	....	....
Allegheny No. 1	3 795	66 000		1.18	32.8
Allegheny No. 2		112 000			
Allegheny No. 3		61 000			
			239 000	....	....
Total, Allegheny Basin	8 023		969 000	2.26	69.2
Monongahela Basin:					
Youghiogeny No. 2	394	35 600		1.69	5.4
			35 600	....	....
Cheat No. 1	1 399	129 000		3.97	19.1
Cheat No. 2		168 000			
			297 000	....	....
West Fork	366	63 500		3.25	5.0
			63 500	....	....
Total, Monongahela Basin	2 159		396 100	3.42	29.5
Total, both basins	10 182		1 365 000	2.53	53.8

* For location, see Fig. 15.

and communities. Farther up stream there are sites where a branch railroad can be shifted to an adjacent valley, or where a main railroad can be elevated above a proposed reservoir flow line, although at enormous expense. These considerations are of paramount importance in the selection of reservoir sites.

Of the seventeen reservoirs recommended by the Flood Commission of Pittsburgh, thirteen were located in the Allegheny basin and four in the Monongahela basin. A summary of data regarding these reservoirs is given in Table 8.

Among the excellent features of this system of reservoirs may be mentioned the large percentage of controlled area, and the good distribution of sites throughout the basin. The degree of protection afforded by this system agrees well with the economic limit of protection determined by more recent studies. The design and location of the dams made them well adapted to the storage of April, May, and June run-off to increase the minimum stream flow during the dry months of the summer and fall. The wisdom of not attempting to develop hydro-electric power at the flood-control reservoirs has been confirmed by subsequent studies.

The occasion for subsequent revision of the original seventeen-reservoir plan arose primarily for two reasons:

(1) Among the largest and most effective reservoirs of the system were those on the Clarion and Cheat Rivers. The former river is the second largest tributary of the Allegheny and the latter is the second largest tributary of the Monongahela. After 1912 the Associated Gas and Electric Company constructed the Piney Hydro-Electric Plant on the Clarion River, and the West Penn Power Company constructed the Lake Lynn Hydro-Electric Plant on the Cheat River. These Companies, it is said, also own property rights in many or in all undeveloped reservoir sites on the respective rivers and have plans for the future power development of these rivers by chains of reservoirs extending from source to mouth. Among the flood-retarding reservoirs recommended by the United States Engineers since 1928 there are none on the Clarion and Cheat Rivers. This indicates a conclusion that existing and proposed hydro-electric developments have eliminated the possibility of constructing flood-retarding reservoirs on these streams. The writers do not concur in this conclusion. They consider that flood protection for the great industrial district of Western Pennsylvania is of such paramount importance as to justify, if necessary, the use of condemnation proceedings to acquire flood-control-reservoir sites on the Clarion and Cheat Rivers. Such proceedings although conducted in the public interest, would give full consideration to the legal rights of the Power Companies. The reservoirs would be among the most valuable and effective ones of the entire system. For some years, the Pennsylvania Water and Power Resources Board has refused to grant permits for further power developments on the Clarion River, because of the fact that public welfare might require the development of one or more sites on this river for flood-control purposes. For these reasons, the writers believe that no final program for flood protection in the Allegheny-Monongahela Valley should be adopted until additional consideration has been given to the development of flood storage-reservoir sites on the Cheat and Clarion Rivers.

(2) The Flood Commission's original plan contained several reservoirs having a storage capacity equivalent to less than 2 in. of run-off from the

controlled water-shed. This capacity is only about one-third or one-fourth of that required to absorb the entire run-off of the water-shed during a great flood. Advances in the art of flood-control engineering have made it evident that the storage of a retarding reservoir can be designated as of "superior" nature only if the reservoir is capable of absorbing completely, or almost completely, any flood that may originate above it. Storage of less amount than this must be regarded as of 'inferior" nature, because of the possibility that the reservoir may be operated so as to fill during the early period of a great flood and before the arrival of the crest. For such a reservoir, it is not possible to set up an operating schedule that will avoid this difficulty in a satisfactory manner. If the reservoir is operated for safe control of the largest floods, it will be very ineffective in smaller floods, and *vice versa*. In the light of more recently acquired knowledge it appears best, therefore, to modify the Flood Commission's original plan so as to substitute storage of a superior type for that of an inferior type.

Although the work of the Flood Commission was supported by influential and public-spirited citizens, many years were spent in an unsuccessful attempt to arouse the public to take the necessary legislative and financial steps to bring about the actual construction of the reservoirs. If another disastrous flood had occurred within a few years after the publication of the 1912 report, it is probable that the reservoirs would have been built. However, the great storm of March, 1913, which caused such terrible damage on the lower tributaries of the Ohio River, did not produce excessively high stages on the Allegheny and Monongahela Rivers, and for the following quarter century, no flood of extraordinary nature occurred on these streams.

In 1927, that portion of the work of the Flood Commission dealing with protection by river walls was reviewed and amplified by the City of Pittsburgh. The Department of Public Works prepared detailed plans for a river wall extending along the front of the Golden Triangle and along a considerable length of the banks of the Ohio River within the city limits. These plans proposed the deepening of the rivers by dredging and also provided for terminal wharves, boulevards, and other civic improvements associated with the river walls. Accurate hydraulic studies were made to determine how flood heights would be affected by the proposed improvements. Before the completion of the report on these studies, the work was brought to a close by conditions arising within the City Administration.

Within the past decade the engineering supervision of flood-protection studies in the Pittsburgh region, as in other parts of the United States, has passed from private engineers to the engineers of the United States Army. Moreover, the old idea that each community or group of communities should pay in full for its own flood protection is being cast aside by the public in favor of Federal financing. These considerations have terminated the technical labors of the Pittsburgh Flood Commission.

Through the efforts of the Flood Commission, an appropriation of \$50 000, one-half from the State of Pennsylvania and one-half from the Federal

Government, was obtained in 1924 to re-study the flood-protection problem and to bring up to date all previous studies. This work was delegated to the United States Engineers, who, after making extensive surveys, designs, and investigations, submitted their findings in a report dated 1928. In March, 1937, this report was still unpublished, but it is said to have furnished the physical basis for the several subsequent reports by the United States Engineers on the same subject. It is the understanding of the writers that none of these subsequent reports is based on surveys and studies that can compare in elaborateness and completeness with those of the report of 1928.

The report of 1928 recommended the construction of eleven reservoirs: Eight in the Allegheny basin and three in the Monongahela basin. The location of these reservoirs is shown in Fig. 16, and pertinent data are given in Table 9.

It is to be noted that the total storage of this eleven-reservoir system is almost twice that of the original seventeen-reservoir system of the Flood Commission. The individual dams are much higher, and their storage is definitely of the "superior" type. In the 1928 report it is stated that this

TABLE 9.—DATA ON RESERVOIRS IN ELEVEN-RESERVOIR SYSTEM PROPOSED BY THE CORPS OF ENGINEERS, UNITED STATES ARMY, IN 1928

Reservoirs*	Controlled drainage area, in square miles	STORAGE CAPACITY		Percentage of main basin controlled
		In acre-feet	In inches of run-off	
Allegheny Basin:				
French Creek	541	140 000	4.8	4.66
Allegheny	2 190	930 000	7.96	18.9
Tionesta	483	133 000	5.17	4.16
Red Bank	460	143 000	5.83	3.96
Mahoning	327	79 000	4.53	2.82
Crooked	278	95 000	6.4	2.4
Conemaugh	1 427	283 000	3.72	12.3
Loyalhanna	267	79 000	5.55	2.3
Total, Allegheny Basin	5 973	1 882 000	5.93	51.4
Monongahela Basin:				
Youghiogheny	1 020	340 000	6.2	13.9
Tygart	1 220	327 500	5.04	16.6
West Fork	366	62 500	3.2	5.0
Total, Monongahela Basin	2 606	730 000	5.27	35.5
Total, both basins	8 579	2 612 000	5.72	45.3

eleven-reservoir system would reduce future flood damage to a negligible amount on the Allegheny and Monongahela Rivers and on a limited reach of the Upper Ohio River. The estimated total cost was $96 378 000. The proposed dams would be equipped with manually controlled gates to permit the conservation of the spring run-off. No hydro-electric power development is contemplated at the reservoirs.

In studying this system, the following comparison between certain features of the Allegheny and Monongahela Rivers, will be of interest:

Comparison of Head-Waters.—

Allegheny River.—There is a favorable site for a large reservoir on the head-waters of the main river (Allegheny Reservoir, 10 miles above Warren,

Pa.). It contains no important towns or industries, and a single-track railroad running through it can be diverted to another valley.

Monongahela River.—There is a favorable site for a large reservoir on the head-waters of the main river (Tygart Reservoir, 3 miles above Grafton, W. Va.). It contains no important towns or industries, and a single-track railroad running through it can be diverted to another valley.

Comparison of Largest Tributaries.—

Allegheny River.—The Kiskiminetas River, its largest tributary, enters a short distance above Pittsburgh, Pa., and has an extremely important effect on floods. The valley is completely occupied by a main railroad line with heavy traffic (Pennsylvania Railroad Freight Line). A favorable site for a large reservoir is 1 mile above Saltsburg, Pa. A railroad relocation above the proposed reservoir level is physically feasible, but would be enormously expensive.

Monongahela River.—The Youghiogheny River, its largest tributary, enters a short distance above Pittsburgh and has an extremely important effect on floods. The valley is completely occupied by main railroad lines with heavy traffic (Baltimore and Ohio and Western Maryland). A favorable site for a large reservoir is 2 miles below Confluence, Pa. A railroad relocation above the proposed reservoir level is physically feasible, but would be enormously expensive.

Next Largest Tributary.—

Allegheny River.—The Clarion River, the second largest tributary, enters at a moderate distance above Pittsburgh. It has been considered unavailable for flood control because of an existing large hydro-electric plant (Piney), with other reservoir sites controlled by a power company. The writers do not concur in this conclusion. A flood-storage reservoir on this river would be of immense value in a flood-control system.

Monongahela River.—The Cheat River, the second largest tributary, enters at a moderate distance above Pittsburgh. It has been considered unavailable for flood control because of an existing hydro-electric plant (Lake Lynn), with other reservoir sites controlled by a power company. The writers do not concur in this conclusion. A flood-storage reservoir on this river would be of immense value in a flood-control system.

Smaller Tributaries.—

Allegheny River.—Favorable reservoir sites on smaller tributaries (French Creek, Tionesta Creek, Red Bank Creek, Mahoning Creek, Crooked Creek, and Loyalhanna Creek), but the total storage of these reservoirs is much less than is desirable for the general plan.

Monongahela River.—A favorable reservoir site exists on one smaller tributary (West Fork), but its total storage is much less than is desirable for the general plan.

In 1929 and 1930, the Consulting Engineers of the Flood Commission of Pittsburgh, Ross M. Riegel and H. A. Thomas, Members, Am. Soc. C. E., reviewed the 1928 report of the United States Engineers, and commended the

excellence of the engineering surveys, designs, and cost estimates described therein. They called attention, however, to the fact that a reservoir system similar to that described in the report, but of reduced capacity, could be built at a cost of about $58 000 000 (that is, at about 60% of the cost of the eleven-reservoir system), and would be capable of eliminating 87% of the flood damage. They concluded that the expenditure of an additional $38 000 000 to eliminate the last 13% of flood damage would be an uneconomic investment. The concurrence of the United States Engineers in this general conclusion is indicated by the selections made in 1936 of reservoirs for construction under Federal appropriations.

Messrs. Riegel and Thomas showed that the uneconomically high storage capacity and cost of the eleven-reservoir system can best be reduced by eliminating certain of the reservoirs, or by constructing smaller reservoirs at points farther up stream. In general, it is not desirable to decrease the heights of the dams, since this would change the nature of the storage from superior to inferior. It is generally agreed that the first step toward cheapening the system should be the elimination of the proposed large reservoir on the Youghiogheny. Although this reservoir would be of great value for the protection of the Pittsburgh District, its cost is saddled with an enormously expensive railroad relocation.

The reservoir on the Tygart River is now (1937) under construction, and it appears certain that some of the other reservoirs will be built in the near future. The decision as to which are to be started first is yet to be made. The Flood Control Act of 1936 provides that the Federal Government shall carry the construction costs of the authorized projects, while local interests must provide without cost to the United States all lands, easements, and rights of way (within certain limits) necessary for the construction of the project. Legislation providing for the payment of that portion of the cost assessed to the local interests has been enacted by the Legislature of Pennsylvania.

Pymatuning Reservoir

Experience with the recently completed Pymatuning Dam demonstrates the effectiveness of the reservoir method of flood control under conditions encountered on the rivers of Western Pennsylvania. This dam was constructed by the Commonwealth of Pennsylvania to provide an adequate supply of water for manufacturing and domestic use during periods of low discharge on the Shenango River. It is located 86 miles north of Pittsburgh and controls a drainage area of 160 sq miles. The capacity of the reservoir is 8 400 000 000 cu ft and its total cost was $3 750 000.

The first year's operation of the Pymatuning Dam has proved its effectiveness in improving dry-weather flow and in controlling floods. The industrial city of Sharon, Pa., is located 25 miles below the dam, at a point where the drainage area is 640 sq miles. On July 1, 1935, the discharge of the Shenango River at Sharon dropped to 9 cu ft per sec. Although the Pymatuning

Reservoir was only partly filled at this time, the gates were opened and sufficient flow was released to supply the water needed for industrial and domestic purposes in Sharon, Newcastle, and other manufacturing communities. This action prevented the complete shut-down of many important industries from July 1 to December 1, 1935.

Since its completion the Pymatuning Dam has prevented two floods. Shortly prior to January 26, 1937, the level of Pymatuning Lake was 2 ft below the spillway. About that time the occurrence of warm weather and rainfall produced floods on all the rivers of the region, the stage at Pittsburgh, for example, being 34.2 ft on January 26. On this day the level of Pymatuning Lake rose to 0.86 ft over the spillway. Because of this storage of peak run-off, the maximum flood stage at Sharon on January 26 was only 0.36 ft.

When the great storm preceeding March 18, 1936, swept over the Middle States and New England, the Pymatuning Reservoir was not full. The gates were closed and impounded the flow so that no flood damage occurred in the Shenango Valley, and no industries were shut down on account of high water. This was the only river valley in Pennsylvania which did not suffer damage during this flood period. Even at Newcastle, 21 miles below Sharon, where the drainage area is 804 sq miles, there were no serious flood losses.

The owners of important industries in the Shenango Valley claim that the Pymatuning Dam project paid for itself during the first year of its life. In general design and construction this dam is similar to those proposed for flood-control on the Allegheny and Monongahela Rivers.

Theoretical Considerations

No reference has thus far been made to the exact effect the various proposed reservoir systems would have on a particular flood. The 1912 report of the Pittsburgh Flood Commission and the 1928 report of the United States Engineers both contain the results of extensive computations purporting to answer such questions. However, a close study of the different methods used in these two computations indicates that in neither case is it possible to have confidence that the final results are accurate. The fact is that, in the present state of the science of hydraulics, no accurate, reliable, and practical analytical method exists for determining the manner in which flood waves from several tributaries will combine to form a single flood wave on a main river.

From considerations based on the volumes under the hydrographs of the tributary floods, together with approximate estimates of the velocity of crest movements, it is possible to form a rough idea of how the tributary waves combine into a single main wave. This was essentially the method used in the 1912 study of the Flood Commission and was the best or only method known to the Engineering Profession at that time. It is, however, subject to errors of large or uncertain magnitude.

In more recent years, a method of flood-wave routing, usually known as the Watkins method, was developed in studies on the Tennessee River under

Colonel L. H. Watkins[25]. This method was used by the United States Engineers in their 1928 studies for flood protection of the Pittsburgh District. It assumes the river to be divided into several reaches extending between successive gaging stations, and the flood period to be divided into several intervals, each ordinarily of 24-hr duration. Each reach during each interval is made to satisfy certain requirements of inflow, outflow, and storage. It has been shown by one of the writers[26] that the errors and uncertainties of this method, as it is ordinarily applied, are probably as great as or greater than those of the simpler approximate method used in the 1912 report of the Flood Commission.

Since little has been published regarding the fundamental hydraulic principles of flood-wave routing, and since a knowledge of these principles is of vital importance in a study of flood protection by reservoirs on a river system the main streams of which are several hundred miles long, it is appropriate to include a brief discussion of the theory at this point. At any cross-section of a river, let z = the elevation of the water surface above datum; v = the mean velocity of the water; t = the time; g = the acceleration of gravity; x = the horizontal distance from the source of the river or from some arbitrary origin; d = the mean depth of the water (cross-sectional area divided by surface width); and s = the hydraulic slope, or loss of head per foot of travel.

The ordinary law that the hydraulic slope of an open channel varies as the square of the velocity is then expressed by the formula, $s = k\,v^2$, in which k is the "conveyance factor" of the channel, involving the effect of both the hydraulic radius and the surface roughness. At any given cross-section, k is a function of z. In general, v and z are functions of both x and t.

In the previously mentioned *Bulletin*[27] it is shown that at any cross-section of a river, the flow must satisfy two conditions: (1) The law of conservation of energy; and (2) the law of continuity or conservation of matter. The differential equations expressing these two laws, respectively, are as follows:

$$\frac{\partial z}{\partial x} + \frac{v}{g}\,\frac{\partial v}{\partial x} + \frac{1}{g}\,\frac{\partial v}{\partial t} + k\,v^2 = 0 \qquad \text{..................(1)}$$

and,

$$d\,\frac{\partial v}{\partial x} + v\,\frac{\partial z}{\partial x} + \frac{\partial z}{\partial t} = 0 \qquad \text{.....................(2)}$$

In Equation (1) the names of the terms, in order, are: The "surface slope term", "velocity head term", "acceleration head term", and "hydraulic friction term." In Equation (2) the names of the terms, in order, are, the "prism storage term", the "wedge storage term", and the "rate of rise term."

The derivation of the Equations (1) and (2), being based on fundamental mechanics, is rigidly exact, but the actual integration to obtain numerical solutions of practical problems is by far too intricate and laborious to be

[25] H. R. Doc. 185, 70th Cong., 1st Session, Appendix B, Pt. II, pp. 40–52.

[26] "The Hydraulics of Flood Movements in Rivers", by H. A. Thomas, M. Am. Soc. C. E., *Engineering Bulletin*, Carnegie Inst. of Technology, Pittsburgh, 1934.

workable. To obtain a solution that is workable from the standpoint of human patience and endurance, it is necessary to replace the infinitesimals, ∂x, by finite reaches of the river expressed in miles, and the infinitesimals, ∂t, by finite time intervals expressed in hours. Further, the necessity of labor saving requires that these finite space and time intervals must be made very coarse.

Unfortunately, this coarseness utterly destroys the accuracy and reliability of the computations. Moreover, to keep the intricacy of the computations within the resolving power of an ordinary human mind, it has been the practice to neglect some of the terms of the fundamental differential equations. For example, the "Watkins method" neglects the second and third terms of Equation (1) and the second term of Equation (2). The fact that the neglected terms may be of great actual importance adds to the fundamental unreliability of these ordinary analytical methods of flood-wave routing. In the writers' opinion, the results of long and tedious computations by these methods are apt to be less accurate and reliable than the results of approximate estimates based on simple assumptions of the type used in the 1912 report of the Flood Commission.

The successful use of mechanical integration in other fields of scientific or engineering practice suggests that it might be possible to construct some integrating machine which would solve the fundamental differential equations of flood motion by a mechanical process. At the Carnegie Institute of Technology, much study has been devoted to the design and operation of devices suitable for this purpose. The most successful machine has been a model channel, of length suitable for indoor construction and with a greatly exaggerated vertical scale, so that depths of flow can be read with the necessary precision. The term, "integrating model", is used to designate a device of this type. Curves of the river are not represented in such a model, the effects of curve friction being taken care of in the roughness adjustment. The construction of each cross-section of the channel of an integrating model to duplicate the area-elevation curve at the corresponding section of the prototype offers only mechanical difficulties. The principal new technical feature involved in the construction of integrating models is the reproduction of the hydraulic friction effect as represented by k.

In the models thus far constructed, the hydraulic friction effect has been produced by sheet-metal baffles extending into the channel from the bed and sides. A technique has been developed for designing these baffles so as to make k vary with z in the model channel in any desired manner[27]. By this process the rating curves of the river can be reproduced in the integrating model, thereby satisfying all the requirements to make the model reproduce the flood-wave hydrographs of the prototype with perfect fidelity.

In such a model of a river system, each important tributary has its own cam-controlled inlet, by which the hydrographs of the tributary floods are reproduced. Several preliminary models representing parts of river systems

[27] "Model Studies of Flood Wave Movement in the Rivers of the Upper Ohio Watershed", by John W. Hackney, Thesis submitted in partial fulfillment of the requirements for the degree of Master of Science, Carnegie Inst. of Technology, 1935.

and involving these principles have been built and tested at Carnegie Institute of Technology, and in 1936, the Pittsburgh Office of the Corps of Engineers recommended construction of a large integrating model of the entire Allegheny-Monongahela River System. At present, work on the construction of this model is under way.

This model will provide an accurate and reliable method of determining how much various flood crests will be lowered by the retarding reservoir system. It will also be used for flood-forecasting. Provided a sufficient number of automatic recording gages are established in the field, and properly manned, this will make possible early and accurate forecasts of high river stages.

AN IDEAL ORGANIZATION FOR THE RIVER AND FLOOD SERVICE OF THE UNITED STATES WEATHER BUREAU

By Montrose W. Hayes,[28] Esq.

Synopsis

Flood forecasts may be based on river-gage relations, or on rainfall data, or on both. The demand for flood forecasts has grown rapidly, and the standard of refinement insisted upon, is constantly increasing. This makes necessary the establishment of additional observation stations and the development of more precise methods of prediction. A plan for improving the river and flood service of the United States Weather Bureau is presented.

Flood forecasting on a systematic basis seems to have had its beginning in France in 1854. The extent of the network of stations used is not known, nor is it clear, from the literature available to the writer, what degree of success was attained in making the predictions. However, the obstacles that undoubtedly were in the way were overcome, for the flood-forecasting service in France has not been interrupted since that beginning. Similar work was undertaken in Italy and Bohemia about 1866, and in the United States in 1871, the latter date being one year after the establishment of the National Weather Service.

At present, two distinctly different methods are used by the Weather Bureau in forecasting flood stages. The older and more refined method is based on gage relations and, more recently, on discharge data. The latter method utilizes largely, or altogether, reports of rain that has fallen or is expected to fall.

In forecasting from gage relations and discharge data, intervening rains must be given weight, but the underlying factors are up-river stages. It may seem unnecessary to state that the farther a reach is from its source water, the greater the time range that is possible in making a flood forecast, but the statement is made to bring out more forcefully the difference in the two general methods of forecasting. On the larger streams, or on the reach of any stream several days removed from the regions of the flood-producing rains, there is sufficient time to make the final forecast in refined terms, both as to stage and as to time. This is done now in a satisfactory manner and has been done for as long as 64 yr. The term of the forecasts ranges from 2 or 3 days in the upper valleys to as many as 3 or 4 weeks in the Lower Mississippi basin.

As a rule, it is impracticable to use gage relation or discharge data in making flood forecasts for regions into which flow numerous small streams,

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that may be considered source-water streams. The gaging of these numerous streams, and of the channels that are water carriers only during heavy rains, is not economically practicable. In general, moreover, the results would be of little value in forecasting. Therefore, it becomes necessary to rely upon reports from rain-gage stations, distributed over the catchment basins in such a way as to provide adequate information on the amount and intensity of the rain, and its geographic distribution. In much of the country east of the Appalachian Mountains, flood forecasts to be timely enough to be of real benefit must be made from rainfall reports.

The time when the work of the Weather Bureau was principally of academic interest has long since passed, and for many years the field offices, as well as the Central Office, in Washington, D. C., have had difficulty in meeting the calls made on them for general weather information. At offices situated on rivers the demands for river stage and flood forecasts have grown concurrently with the requirements for other service, and as the use of rivers increases these demands will continue to grow. The development of streams for power, navigation, and irrigation, the larger consumptive use of water by growing cities, and the greater use of the flood-plains by industry, all make flood forecasts not only desirable but absolutely necessary. Too, contrary to popular opinion, the successful operation of flood-control works, as well as their construction, requires forecasts of ice-forming weather, of impending rains, and of floods.

The standard of refinement of flood forecasts is set by those who use them. This statement may cause some surprise, but its truth can easily be demonstrated. If the interests along a river can be protected by 2-day forecasts verified in stage with an accuracy of about 2 ft, it would be useless to spend the money necessary to provide 3-day forecasts with an order of accuracy of less than 0.5 ft. On the other hand if the latter and more accurate forecasts were needed, an effort would be made to obtain the money that would make them possible. Because of the unprecedented heights of the floods of March, 1936, when, in inundated cities, each 6 in. of rise meant an additional loss of enormous extent, the Weather Bureau now finds itself facing unprecedentedly widespread demands for river stage and flood forecasts of longer range and greater refinement. In providing them many obstacles will have to be overcome. Changes in the present plan of operation will be necessary, and some outstanding deficiencies must be met. The simpler requirements need not be reviewed here. Others, more complex or difficult of attainment, are presented to show why the Bureau can not immediately strengthen its river service to the extent known to be desirable. These requirements are as follows:

(*a*) The establishment of more and better placed rainfall stations, especially in head-water regions.

(*b*) The installation of an adequate network of recording rain-gages to enable the forecaster to know the intensity of the rainfall. At present, most of the rain-gages used in flood work are of the eye-reading, 8-in. type.

(*c*) Surveys of the amount and condition of snow in the eastern mountains. Few data of this kind are available at present. Reliable and prompt rainfall reports are not sufficient when the mountain regions hold a great quantity of water in the form of snow, which is likely to be released by the rain.

(*d*) Arrangements for a more reliable transmission of rainfall and river-stage reports from the sub-stations to the district centers. In the eastern floods of March, 1936, the failure of wire communication was almost entirely responsible for any lack of timeliness and accuracy in the warnings issued. Except in very unusual cases the telephone and telegraph wires answer all purposes with a great degree of satisfaction, but it is in the unusual cases—the emergencies—that the greatest need for the reports exists. A river forecaster without information from the drainage area above him is helpless. The problem is difficult to solve. It has been suggested that radio stations should be established in flood-producing regions that are without reliable wire communication, and that they should be manned by Weather Bureau employees to transmit reports promptly under all conditions, to the forecasting center. This, of course, would be a solution, but it is believed that the cost would not be justifiable; certainly, it would not be, until it could be

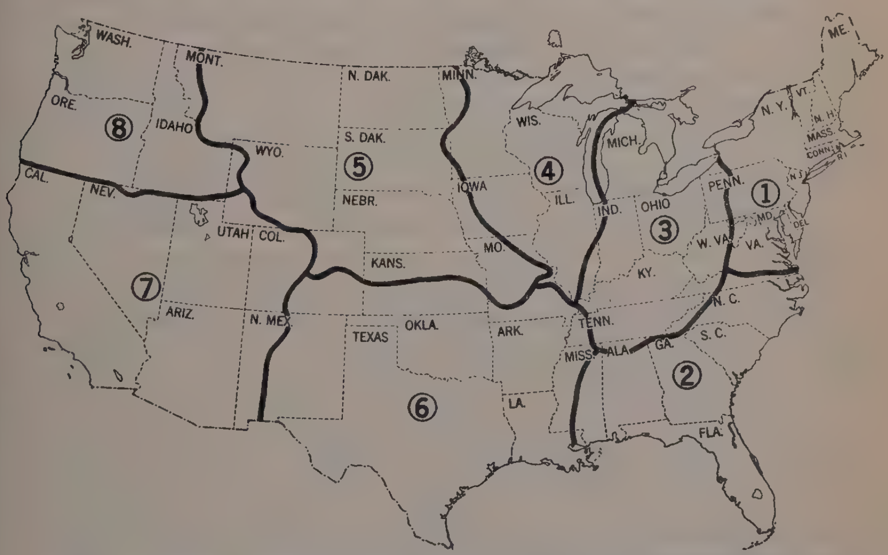

FIG. 17.—PROPOSED DIVISION OF THE UNITED STATES FOR FLOOD-FORECASTING SERVICE.

shown definitely that the Weather Bureau had exhausted every other and less costly means for having reports transmitted satisfactorily. One plan now (1937) being investigated would utilize amateur radio operators to transmit reports in times of emergency. Another proposal is to find hydrologically satisfactory locations for rain-gages, and for some river gages, from which reports can be sent by automobile to telegraph or telephone offices. This latter procedure would be expensive, but considerably less expensive than the maintenance of Weather Bureau radio stations.

(*e*) Division of the country into eight sections for river administrative and forecasting purposes (Fig. 17), and the establishment of a staff of men in each section, under a section head, to handle the river work. These men should be charged with: (1) Placing and supervising the operation of the sub-stations; (2) developing and putting into effect a plan for transmitting reports to the forecasting centers; (3) co-ordinating all phases of the work, and co-operating with other organizations engaged in river work; (4) investigating the requirements for forecasts and arranging to meet those requirements; and (5) developing formulas for forecasting. Through a close co-operation with the U. S. Geological Survey, the Weather Bureau is now obtaining discharge data on most of the rivers of the country. This information can be used to advantage, in combination with the data supplied by the Weather Bureau itself, in developing more accurate formulas for river-stage and flood forecasts. However, the officials in charge of the Weather Bureau stations, who have multitudinous duties to perform, have not the time to devote to all the intricacies of the river work. It is obviously necessary to have a number of well-qualified men who can give all their time to studying these data and developing forecasting formulas.

The foregoing list of requirements for a river-forecasting service of a high order of accuracy is not merely something to be desired and never attained. On the contrary, it represents a plan set up by the Chief of the Weather Bureau in 1935. A modest beginning toward its accomplishment was made on July 1, 1936. Small staffs have been placed both in the Missouri Valley and in the Upper Mississippi Valley, and other parts of the country will be taken care of, as it becomes possible to do so.

At present, no definite decision has been reached as to who shall actually make the forecasts. There are sixty-four forecasting centers, and it is thought the usual station personnel can handle the forecasting with the facilities that will be provided by the river and flood staffs. The staffs, however, will be mobile, and in emergencies they will be available to do anything that will aid in issuing timely and accurate warnings of impending floods.

FEDERAL PLANS FOR FLOOD CONTROL

By W. E. R. Covell,[29] M. Am. Soc. C. E.

Synopsis

It was natural that the watercourses and their valleys should have had a great influence in the early settlement, trade, and transportation of the United States since they were the lines of least resistance to the vast unexplored inland country. Later, devastating floods occurred in the valleys and protective measures were sought. The first flood protection sponsored by the Federal Government was in the Mississippi Valley followed by protection in the Sacramento Valley, California. In 1925, Federal interest in river planning of broad scope was manifested, which later resulted in the "308" reports of the Corps of Engineers, U. S. Army, covering the possibilities of navigation, flood control, water power, and irrigation. By June 30, 1935, nine Federal flood-control projects were in force which were followed by twenty-seven additional projects throughout the country to June 22, 1936, when the general Flood Control Act was approved. A Federal policy was thereby established, 219 main flood-control items were authorized in 46 major basins and localities, and 232 localities were designated for further investigation. Fourteen reservoirs were authorized for the protection of Pittsburgh, Pa., and the Ohio Valley, based upon the "308" reports. The general features of the reservoir system, which contemplates control of flood waters at or near their source, are presented.

Introduction

It was natural that the watercourses should have had a great influence in the early settlement, trade, and transportation of the United States. They were the lines of least resistance to the vast unexplored country that lay inland; they provided transportation for the settlers and power for the first mills; and their fertile valleys produced rich crops and afforded topographically favorable town sites.

They were not an unmixed blessing, however. When the valleys were first settled, they were accepted in the condition in which they were found, because the country was new. Later, devastating floods were experienced, which went their way to the sea in the channels which Nature had provided. When the primary banks could not confine the flows they spread to the flood-plains of the inhabited valleys.

Man's ingenuity was aroused, and he sought to protect himself and his property. The progressive development of the valleys was accepted as a necessity and protective measures were early devised to keep the flood-waters

[29] Lt. Col., Corps of Engrs., U. S. A.; U. S. Dist. Engr., Pittsburgh, Pa.

within bounds. To-day such work is still in progress. The extreme floods of recent times and the heavy losses sustained thereby have aroused the country at large as never before to a realization of their effect upon welfare and progress. In the last analysis, economic justification will probably govern the extent to which flood protection will eventually be provided.

Flood Control on the Mississippi River

The largest drainage basin of the country is that of the Mississippi River, with an area of 1 240 000 sq miles. The total area of the Continental United States is 3 025 789 sq miles. This basin includes all or parts of thirty-one States and about 20 000 sq miles of Canada. The first levee along the Mississippi River was built by the French in about 1718. Levees were gradually extended with the growth of settlements, under provisions of the land grants, which required the grantee to construct and maintain a levee line along the river front of his property.

The floods of 1849 and 1850 caused widespread damage in spite of the levees. National interest was aroused by the pleas for Federal aid and resulted in the Swamp Land Act of 1850, which granted to the several States all unsold swamp and overflowed lands within their limits to provide funds to reclaim the districts subject to overflow. In 1850, also, Congress authorized a topographical and hydrographic survey of the delta of the Mississippi River, with such investigation as might lead to the determination of the most practical plan for securing it from inundation. This work was placed in charge of Captain A. A. Humphreys, later assisted by Lieutenant H. L. Abbot, both of the Corps of Topographical Engineers, U. S. Army. Another survey and report of a similar nature, authorized by Congress in the same year, was made by Charles Ellet, a Civil Engineer employed by the Corps of Engineers.

The formation of the Mississippi River Commission was authorized by Congress in 1879. It consisted of seven commissioners of whom three were required to be from the Corps of Engineers, U. S. Army, one from the United States Coast and Geodetic Survey, and three from civil life. Two of the latter must be civil engineers. Between 1890 and 1917, the periodic appropriations made by law for expenditure by the Commission were divided among levees, revetments, and dredging, but the levees were considered as an adjunct to river improvement. It was not until the Flood Control Act of March 1, 1917, that by law, flood control as such became a definite part of the work of the Commission. The Act of 1917 provided for co-operation by local interests which were required to provide the necessary rights of way, pay not less than one-third of the cost of levee construction, and to assume the entire cost of maintenance after completion. The cost of river improvement (revetment and dredging) was to be paid for entirely from Federal funds.

In 1927, the Lower Mississippi River experienced the highest flood of record to that time. More than 200 breaches in the levees occurred, although it is of interest to note that only a few were in levees built by the Commission, and only one in a levee built to the Commission's standard dimensions. Public

interest was aroused by the catastrophe, which submerged about 20 000 sq miles of the valley and cost the lives of about 250 persons. The Flood Control Act of 1928 resulted, based on the report of the late Major General Edgar Jadwin, Corps of Engineers, U. S. Army, M. Am. Soc. C. E., then Chief of Engineers. The project for the valley of the Mississippi River, as outlined in this Act, provides for the control of flood waters from Rock Island, Ill., to the Head of Passes near the Gulf of Mexico, by means of levees. Tributaries and outlets, in so far as they are affected by the Mississippi River, are also included within the scope of the project. (The alluvial valley of the Mississippi River consists of the St. Francis, the Yazoo, the Tensas, and the Atchafalaya basins, as well as the alluvial lands around Lake Pontchartrain.) The plan called for raising existing levees by about 3 ft and strengthening them, and permitting flood waters in excess of the safe capacity of the leveed channel to find their way to the Gulf through the natural floodways in the lowlands adjacent to the river, which have always carried the waters of extraordinary floods. Levees were planned on the edges of these natural floodways, to restrain the waters within them as far as practicable and to protect the maximum area of land that it was economically justifiable to reclaim. The flood waters would not escape into the floodways until they reached a dangerous stage in the main leveed channel.

An appropriation of $325 000 000 was authorized by the Act of 1928 for carrying on this work. The Act further provided for local co-operation in the project to the extent that States or levee districts were required to maintain the flood-control works, except the controlling and regulating structures, after their completion; provide all the necessary rights of way without cost to the United States; and contribute 33⅓% of the cost of the project work between Rock Island, Ill., and Cape Girardeau, Mo. A later Act, approved April 23, 1934, provided for the reimbursement of States or local levee districts for the cost of the rights of way or easements required for the levees which the Federal Commission was obligated to provide under the Act of May 15, 1928, provided the cost was reasonable.

The general project for flood control on the Lower Mississippi River adopted by the Act of May 15, 1928, has been amended by the Flood Control Act of June 15, 1936[30], and modified in accordance with the report of February 12, 1935, of the Chief of Engineers, U. S. Army[31]. The adopted modifications of the project consist of the following: (1) The abandonment of the Boeuf Floodway, and in lieu thereof the construction of the Eudora Floodway (west of the Mississippi River, and extending from the latitude of Eudora, Ark., into the Red River back-water area), with a control structure at its head; (2) the construction of a rear protection levee extending from the head of the Eudora Floodway north to the Arkansas River, located so as to afford adequate space for the escape of flood waters without endangering the levees on the east side of the river; (3) the maintenance of the present river levees between the head of the

[30] Public Doc No. 678, 74th Cong.

[31] H. R. Committee on Flood Control, Doc. No. 1, 74th Cong.

Eudora Floodway and the northern junction with the protection levee, at the 1914 grade and section, except in front of densely populated areas, as a part of a ring levee; (4) the construction of a floodway extending from the Mississippi River north of Morganza, La., to the Atchafalaya River back-water, with a control structure at its head; (5) the raising of the levees from the head of this floodway to the head of the Atchafalaya River to full standard grade and section; (6) the immediate completion of the guide levees in the Atchafalaya basin to afford full protection to all lands outside of these levees; (7) the construction of an additional outlet to the Gulf of Mexico, west of Berwick, La.; (8) the increase of the discharge capacity of the leveed channel of the Atchafalaya River and of its outlets; (9) a six-year program for the improvement and regularization of the Mississippi River, including the continued maintenance of the navigation channel provided in the previous project; and (10) the flood control of the St. Francis and Yazoo Rivers.

It was contemplated that the unfinished parts of the levees and structures authorized by the Act of 1928 and not modified by the Act of May 15, 1936, would be completed as planned. The total estimated cost of all works added to the previous project, including the acquisition or reimbursement for land rights, was \$313 000 000, or \$245 000 000 in excess of the unappropriated balance of the appropriation previously authorized.

Federal Interest in Other Streams

The first expansion of Federal interest in flood control to another locality of the United States was authorized by the Flood Control Act of March 1, 1917. It resulted in a general project for flood control, including channel enlargement, cut-offs, by-pass weirs, outfall gates, and levees, for the Sacramento River, in California. The project, later modified by the Flood Control Act of May 15, 1928, represents the outgrowth of the early activities of the California Débris Commission created by an Act of Congress in 1893, and consisting of three officers of the Corps of Engineers, U. S. Army. The Act of 1917 required that the State of California share equally with the United States in the cost of the works, and in addition, that all rights of way, easements, and lands be provided free of cost to the Federal Government. The Flood Control Act of May 15, 1928, required the State or other local interests to contribute one-half the cost of constructing the levees to Commission standards and to complete the project.

In 1925, Federal interest was manifested in river planning of a broad scope. The River and Harbor Act approved on March 3 of that year directed the Secretary of War and the Federal Power Commission jointly to prepare and submit to Congress an estimate of making such examinations, surveys, or other investigations, as in their opinion were required for those navigable streams of the United States, and their tributaries, whereon power development appeared feasible and practicable, with a view to formulating general plans for the purposes of navigation and for the prosecution of such improvements in combination with the most efficient development

of the potential water power, the control of floods, and the needs of irrigation. The streams and their tributaries to be investigated were designated in a Federal report[32], and the River and Harbor Act of January 21, 1927, directed the Chief of Engineers to prosecute the investigations. The resulting reports have become quite generally known as the "308" reports.

The work covered practically all the principal drainage systems of the United States. The surveys first authorized, involved an estimated cost of $7 322 400. In addition, the Flood Control Act of May 15, 1928, authorized the expenditure of $5 000 000 of flood-control funds for surveys of tributaries of the Mississippi River. In particular, the Chief of Engineers was thereby directed to make a comprehensive and detailed investigation of the possible alleviation of Mississippi River flood conditions by means of reservoirs, operated either primarily for that purpose or for some other purpose or combination of purposes, on tributaries of the Mississippi that were included in the "308" list. Further study or review of the "308" reports is authorized by Section 6 of the River and Harbor Act of August 30, 1935. By June 30, 1935, a total of $10 254 123.13 had been allotted. The work was conducted by the Corps of Engineers, through thirty-six of the forty-three District Offices. The resulting reports, together with other reports by the Corps of Engineers, furnished the basis for the flood-control projects adopted by the Flood Control Act of June 22, 1936. In addition to flood control proper, the possible influence of potential water power and irrigation developments on flood control has been covered in the "308" studies.

It is interesting to note the status of Federal flood control in the United States and Territories as of June 30, 1935. Under Federal Public Works and Emergency Relief programs, nine flood control projects were in force, seven of them in various parts of the United States and two in Alaska, in addition to the initial Mississippi and Sacramento River projects. These projects were: Winooski River System, Vermont (reservoir system); Wallkill River System, New York (channel improvement); South Canadian River, New Mexico (reservoir); New River, West Virginia (reservoir); Tygart River, West Virginia (reservoir); Muskingum River System, Ohio (reservoir system and channel improvement); Los Angeles and San Gabriel Rivers, California (flood-control system); Salmon River, Alaska (dikes and clearing); and Lowell Creek, Alaska (dam). The total estimated cost of these projects was more than $100 000 000, including the cost of construction and land and damages. Most of them required local co-operation, principally with respect to the payment for land and damages.

Between June 30, 1935, and the approval of the general Flood Control Act of 1936, a total of twenty-seven additional flood-control projects was approved for construction under provisions of the Emergency Relief Appropriation Acts of 1935 and 1936. These projects were scattered throughout the country and consisted of levee, channel, cut-off, flood-wall, and reservoir improvements at a total estimated cost of about $22 351 000. Payment for lands and damages was the responsibility of local interests. Notable projects

[32] H. R. Doc. No. 308, 69th Cong., 1st Session.

in this list were the Sardis Reservoir, in the Yazoo River basin, Mississippi (a part of the modified plan for Mississippi River flood control); and the Possum Kingdom Reservoir, on the Brazos River, Texas (one of the thirteen reservoirs planned by the Brazos River Conservancy and Reclamation District).

The Flood Control Act of 1936

The Flood Control Act of June 22, 1936, opens a new era for flood control in the United States. For the first time in the history of the nation, a Federal policy with respect to flood control for the country at large is adopted by law. The first four sections and Paragraph 1 of Section 5 of the Act are contained in the Appendix.

Analysis of the provisions of the Flood Control Act discloses the followin general features: (*a*) Flood-control projects are authorized in 46 major basins and localities throughout the United States; (*b*) the Act contains 219 main flood-control items, some of which involve several sub-items; (*c*) the types of projects include channel improvements, dikes, reservoirs, levees, cut-offs, diversions, floodways, and flood walls; and (*d*) the estimated total cost of the projects authorized is from $396 532 880 to $397 532 880, of which $300 571 325 is for construction and from $95 961 555 to $96 961 555 is for land and damages. In addition to the authorization of definite flood-control projects, the Act provides for preliminary examinations and surveys for flood control at 222 localities throughout the United States, and for surveys, studies, and reports at 10 Southern and Western localities where there are indications that opportunities may exist for useful flood control in combination with hydro-electric power development whenever sufficient markets become available to absorb the power.

The Act of 1936 does not repeal the Flood Control Act of 1928, or any provision of any law amendatory thereof. The sum of $310 000 000 is authorized to be appropriated for the flood-control projects adopted by the Act. Not more than $50 000 000 may be expended during the fiscal year ending June 30, 1937, and provision is made for relief of unemployment, in constructing the projects.

The Flood Control Act of 1936 represents national stream planning on a large scale. The most meritorious projects of the country at large are included in the initial program of development. The further possibilities are recognized. The ultimate development will likely be determined by economics of cost and benefit.

The Federal policy with respect to local co-operation in projects has been traced in this paper from the original flood-control project on the Mississippi River to the present time. Such co-operation has largely attended the development of flood control and is a feature of the Act of 1936 which provides that when the cost of the land, rights of way, and damages exceeds the construction cost of a project, the resulting participation of the Federal Government and the local interests is on an equal basis. Local interests are required to maintain and operate all the flood-control works after completion.

The project for the Ohio River basin authorized by the Flood Control Act of June 22, 1936, provided for two features, namely:

"Reservoir System for the Protection of Pittsburgh: Construction of reservoirs for the Allegheny-Monongahela basin as in comprehensive plan for the protection of Pittsburgh and for the reduction of flood heights in the Ohio Valley generally, as set forth in House Document Number 306, 74th Congress, 1st Session, and in the report on the Allegheny-Monongahela Rivers and tributaries on record in the Office of the Chief of Engineers; estimated construction cost, $20 646 000; estimated cost of lands and damages, $34 569 000.

"Reservoir System for the Reduction of Ohio River Floods Below Pittsburgh: Construction of reservoirs including the completion of the Bluestone Reservoir now under way, which together with the reservoirs for Pittsburgh flood control, constitutes a comprehensive plan for flood control on the main stream of the Ohio River and on the tributary stream below the reservoirs, as set forth in House Document Number 306, 74th Congress, 1st Session; estimated construction cost, $19 616 800; estimated cost of lands and damages, $10 519 600."

The report mentioned as being on record in the Office of the Chief of Engineers is an unpublished report, and House Document No. 306, 74th Congress, 1st Session, is a published report, both prepared in accordance with House Document No. 308, 69th Congress, 1st Session.

Projects in the Ohio Basin

The Ohio River is formed by the confluence of the Allegheny and Monongahela Rivers at Pittsburgh, Pa., and flows in a general southwesterly direction 981 miles to enter the Mississippi River at Cairo, Ill. The river discharge varies widely, the maximum and minimum at the mouth being 1 600 000 (prior to January, 1937) and 22 000 cu ft per sec, respectively. The basin, which embraces parts of fourteen States, is 800 miles long and has an area of approximately 204 000 sq miles. The topography is greatly diversified. In the northern and western areas the surface is generally flat to rolling, whereas in the eastern and southern areas it is rugged and mountainous. The normal annual precipitation for the basin is 45 in. varying from 83 in. in the southeastern mountainous part to 31 in. in the northwestern part.

Because of its large and extensively developed coal fields, the basin is one of the principal industrial regions of the United States. The outstanding development is that of the iron and steel industry, centered at Pittsburgh, Pa., and extending up the Monongahela River and down the Ohio River. Cincinnati, Ohio, Louisville, Ky., and Wheeling, W. Va., are other large centers of industry on the river. Outside the industrial regions, agriculture and coal mining are the principal activities.

High stages occur frequently on the Ohio River and its tributaries, the range in stage from low water to high water attaining 70 ft (prior to January, 1937) in some reaches of the main river. The bottom-lands, however, are generally narrow, and most of the cities, roads, etc., are above the ordinary flood stages. Nevertheless, between 1876 and 1936 twenty-three floods

have risen 5 ft, or more, above ordinary flood stage, with damage to cities, towns, industries, railroads, and highways. A large part of this damage is due to the flooding of basements and the lower floors of buildings, with consequent injury to goods and furniture therein. Crop damages are of less importance. Prior to the flood of March, 1936, the average annual flood loss on the main stem of the river, exclusive of Pittsburgh, was estimated at $1 860 000. (About two-thirds of this loss is at or above Cincinnati, because of the large industrial development along the upper river, and the relatively greater flood heights there experienced.) It was also estimated that an effective system of flood control would afford an annual benefit of about $1 000 000 to the lands bordering the Ohio River, because of increased value for urban or farm use.

Reports have shown that the construction of levees or walls for the protection of towns and cities on the Ohio River against high floods appears to be economically justified in only certain cases. The cost of levees for the protection of agricultural lands would greatly exceed any possible benefits.

Because of the intensive development of the valleys, the construction on the main rivers, of dams of sufficient height to afford flood storage and effective power development would inflict widespread and costly flowage damage. For the control of floods in the Mississippi Valley, the most effective location for such a dam would be near the mouth of the Ohio River, but investigation of the most practicable site, has shown it to be inadvisable. The same is true of other possible sites in the more highly developed upper parts of the Ohio Valley. Flood control on the Ohio River, therefore, must primarily be obtained by works on its tributaries.

A compilation of the most favorable plans for the comprehensive development of the water resources of the Ohio River basin, as developed in the "308" surveys, shows that about 250 water-power developments, supplemented by 32 flood-control reservoirs, and levees and channel improvements on tributaries, may eventually be warranted. The total installed hydro-electric capacity of such a comprehensive system would reach 8 600 000 kw. So large an amount of power could not be absorbed for many years, and, obviously, such a development could only be made progressively, as the growth of the power market justified the individual installations. The total cost of such a comprehensive development is estimated in the "308" reports at $1 700 000 000, and the annual charges at $160 000 000. The possible future annual returns from power are estimated at $169 000 000; the annual benefits from flood control (largely on the tributaries), at $8 600 000; and the annual benefits to water supply and navigation, at $500 000.

A possible comprehensive reservoir plan for the alleviation of floods on the Mississippi River, through works designed for the control of floods on its tributaries was investigated in connection with the "308" reports. It included 81 reservoirs on the tributaries of the Ohio, with a capacity of 25 300 000 acre-ft. The estimated cost of these 81 reservoirs was $517 000 000.

Three large flood-control reservoir projects included in the comprehensive plan are now (1937) under construction in the basin. One of these, on

the Tygart River, a tributary of the Monongahela River, is not only for the purpose of flood control but also to assure an adequate supply of water for the operation of the locks on the Monongahela during periods of extreme drought, such as occurred in 1930. The dam is being built by the War Department with funds allotted by the Public Works Administration. The second project is a system of reservoirs for the control of floods on the Muskingum River and its tributaries, which has been undertaken by the War Department in co-operation with the Muskingum Conservancy District. This project fulfills the purpose of two of the reservoirs contained in the comprehensive plan. The third project is the Norris Dam, on Cove Creek, a tributary of the Tennessee River, nearing completion by the Tennessee Valley Authority.

The Chief of Engineers in 1935 approved the selection of fourteen additional flood-control reservoirs in the Ohio Basin, the immediate construction of which appeared to be warranted. They will be built by the Federal Government after appropriation of funds, provided local interests furnish the necessary lands and flowage, hold and save the United States against damage, and take over the works after completion. These works include nine reservoirs above Pittsburgh, forming the comprehensive plan for the protection of that city, three on the tributaries of the Kanawha, and two on the Licking River, which enters the Ohio opposite Cincinnati. The total estimated cost of the nine dams above Pittsburgh was \$55 215 000, of which \$20 646 000 was for construction and \$34 569 000 was for lands and damages. At one of the project reservoirs on the tributaries of the Kanawha (the Bluestone Reservoir on the New River), power production can apparently be combined with flood control. The construction of this reservoir and appurtenant power installation at an estimated cost of \$13 000 000 was recommended in a comprehensive report on the Kanawha River[33], provided a definite agreement could be entered into for the disposal of the power at prices that would protect the public interest. The total cost of the other two flood control reservoirs on the Kanawha was estimated at \$9 849 000 of which \$2 886 000 was for construction and \$6 963 000 was for lands and damages. The total cost of the two proposed reservoirs on Licking River was estimated at \$7 345 000, of which \$5 046 000 was for construction and \$2 299 000 was for lands and damages. The total estimated cost of the construction of the reservoirs recommended, exclusive of the Bluestone Reservoir, was \$28 578 000, and the total estimated cost of lands and damages was \$43 831 000. The estimates were made at the time the "308" reports were prepared.

The system of fourteen reservoirs, together with the reservoirs now under construction on the Tygart and Muskingum Rivers, may be expected to reduce the peak of a great Ohio River flood, similar to that of 1913, by 7.5 ft at Pittsburgh, 3 ft at Cincinnati, 1.6 ft at Louisville, and about 6 in. at Cairo. Lesser floods, in general, would be reduced to a greater degree. The direct annual benefits from this system of reservoirs were estimated (prior to the great floods of March, 1936, and January, 1937) at \$4 100 000,

[33] H. R. Doc. No. 91, 74th Cong. 1st Session

and the annual carrying charges at \$3 416 000. The flood of March, 1936, is the highest of record in the Upper Ohio River and resulted in new data as to the probable frequencies of high stages. These data indicate that the flood-control benefits will be greater than previously estimated. The drainage area to be controlled by the system of fourteen reservoirs is 15 936 sq miles of the 204 000 sq miles total for the Ohio River basin. The gross capacity of the reservoirs amounts to 3 169 510 acre-ft.

The immediate local effect and benefit of a flood-control reservoir is directly tangible and certain, but its effect on flood heights at the more remote localities on the main stem of the Ohio River is less certain. The just apportionment of the cost of structures of this character between the Federal Government and the local interests presents a considerable problem. The apportionment under which the construction cost would be borne by the United States and the land damages by local interests is considered advantageous, in that it assigns to local interests that part of the cost which they can most effectively undertake.

Prior to 1936, great floods had been experienced at Pittsburgh, notably in 1762, 1763 (the succeeding year), 1809, 1810 (another succeeding year), 1816, 1832, 1884, 1902, and 1907. The flood menace has ever been present, but the risk has been accepted and the flood losses taken as they occurred. In March, 1936, the Pittsburgh area experienced its greatest flood of record; the river reached a stage of 46 ft which was 21 ft above flood stage, 7½ ft higher than the great flood of March 7, 1907, and 12.6 ft higher than that of March, 1913, which was an extremely high flood for the Ohio River as a whole. It had been predicted as the result of engineering studies by the Flood Commission of Pittsburgh published in 1912, that Pittsburgh would some day experience a 43-ft flood. The flood studies of the War Department indicated that a 47-ft flood stage was possible under certain conditions. From 1854, when continuous records began, to the close of 1936, 93 flood stages, or an average of 1.1 per yr, have occurred at Pittsburgh. According to the all-time record dating back to January 9, 1762, the eleven highest floods crested at stages ranging from 35 to 46 ft. The month of March is the most prolific for occurrence of floods at Pittsburgh; January ranks second, February third, and December fourth.

With the benefit of the continuous record of floods over a long period of time, floods of various heights may be given a frequency rating according to the law of probability The flood of March, 1936 (46 ft), may be expected to occur once about every 270 yr, and the flood of March, 1907 (38.5 ft), once about every 26 yr. At first thought, these frequencies appear to lend some comfort to unfortunates situated in the stricken areas. Although it is entirely possible that this generation may never witness a recurrence of the March, 1936, flood, the menace is ever present and high waters may occur close together. This possibility was borne out in 1762 and 1763, when stages of 39 and 40.9 ft, respectively, occurred at Pittsburgh.

The Pittsburgh area lies within the influence of both the Allegheny and Monongahela River Systems, the flood waters of which converge to the head

of the Ohio River. The time of arrival of the flood crests from these river systems, at Pittsburgh, has an important bearing upon the height of the flood stages, which is also influenced by the peculiarities of the distribution of precipitation over the separate drainage areas. The differences in latitude and altitude and related temperatures over the basin are ordinarily redeeming factors, although under some circumstances these factors may become completely nullified. The respective natural peculiarities of the adjoining Allegheny and Monongahela River basins have been instrumental in minimizing the number of floods experienced at Pittsburgh. However, there is always the possibility that conditions may be such that the converging flood waters from the two basins may arrive at Pittsburgh at nearly or at exactly the same time, with disastrous results.

The possibility of flood control by means of reservoirs in the Allegheny-Monongahela basin has been the subject of numerous investigations beginning with that of the Flood Commission of Pittsburgh. The principal object of these flood-control investigations has invariably been to determine the feasibility of the control of floods at or near their source, since it is readily recognized that such control would be productive of the most widespread benefits. In effect, a part of the drainage area would be practically cut off from the remainder by means of flood detention reservoirs. Such reservoirs should have sufficient capacity to retain practically the entire inflow thereto until it can be released so as to reach Pittsburgh after the high water from the intermediate, uncontrolled drainage area above that city has gone on down the Ohio River. As the Monongahela and Lower Allegheny River Valleys cannot advantageously be utilized for reservoirs, resort must be had to locations on the head-waters and the principal tributaries. The individual reservoir sites that have been considered in the investigations have been selected after a study of the physical and economic features of the basin as a whole although, in this respect, the basin offers little choice of sites, and with a view to distributing the individual units so that as many of them as possible would become effective no matter what the particular storm and consequent run-off conditions might be.

As a result of previously mentioned investigations and reports by the U. S. Corps of Engineers, the Tygart River Reservoir was authorized and is now (1937) under construction at an estimated cost of \$15 700 000. This reservoir will have a capacity of 314 000 acre-ft for the control of spring floods, equivalent to a run-off of 5 in. of rainfall from the tributary area of 1 183 sq miles. The reservoir is also designed to provide an adequate low-water discharge for navigation purposes on the Monongahela River. As a flood-control reservoir, it will serve as the initial unit in any flood-control reservoir system which may be provided in the Allegheny-Monongahela River basin.

The project system of flood-control reservoirs for the Allegheny-Monongahela River basin above Pittsburgh adopted by the Flood Control Act approved June 22, 1936, consists of nine units, in addition to the Tygart Reservoir. This system may be considered tentative. Revision of the "308" studies by bringing them up to date may indicate that additional reservoirs or other

facilities may be economically justified. The total capacity of the project system, including the Tygart Reservoir, would be 2 272 500 acre-ft and the total amount of the drainage area controlled would be 7 487 sq miles. The basin above Pittsburgh comprises 19 117 sq miles, which would leave 11 630 sq miles, or 61% of the basin, uncontrolled by reservoirs.

Investigation of the probable effectiveness of the reservoir system indicates that the flood of March, 1907 (stage, 38.5 ft at Pittsburgh), would have been reduced to a stage of 31.5 ft and the flood of January, 1913 (stage, 34.3 ft) to 27.9 ft. For the great flood of March, 1936 (stage, 46 ft), studies indicate that the reduction in stage would have been about 7.7 ft. Slightly reduced benefits resulting from the reservoir system would also extend down the Ohio River to and below Wheeling, W. Va.

The total cost of the nine reservoirs of the system was estimated at the time the "308" reports were made, to be $55 215 000, of which $20 646 000 would be for construction and $34 569 000 for land and damages. It is evident that the Allegheny-Monongahela River basin does not represent virgin territory in which reservoirs may be built cheaply at favorable physical locations. Railroad lines early pre-empted the main valleys in a region of extensive natural resources. In every reservoir site, except one (Crooked Creek), existing rail development would be affected, and in some cases extensive relocation would be required. Numerous small villages would also have to be abandoned.

The benefits that would result from flood control by means of reservoirs should outweigh the cost of providing the facilities. Although a flood of the magnitude of March, 1936, causes widespread damage of very great amount, it is of infrequent occurrence, so that the average annual damage from such floods is relatively small. The floods of small heights, although they have a high frequency, also result in relatively small annual damages. It is the floods of intermediate height; that is, neither extremely high nor just barely flood stage, that result in the greatest amount of average annual damages, since the damage per flood is relatively high and they occur fairly often.

The unprecedented flood of March, 1936, in the Upper Ohio River area presented some new evidence of cause and effect of extreme floods. The project reservoir system, however, was planned to accommodate certain hypothetical floods. These represent the probable results of the translation of large storms of record, such as that of March, 1913, which involved only a part of the Upper Ohio River basin, to more effective positions on the basins. Some adjustment of the reservoir system may be found desirable in view of the March, 1936, flood. This matter is now (1937) under investigation as provided for under the various Acts of Congress. The City of Johnstown, Pa., (in the Allegheny River basin) experienced a devastating flood in March, 1936. Flood-control measures for that area are also under consideration. A reservoir, or reservoir system, above Johnstown would supplement the reservoir system for the protection of Pittsburgh and the Ohio Valley. The existing reservoir system project, however, would have to be modified by Congressional action to include such facilities.

The flood-control work of the Corps of Engineers, U. S. Army, has been and will continue to be prosecuted in obeisance to the mandates of Congress. Flood control is essentially a matter of engineering and economics, and it is recognized that it presents many problems which must be solved effectively, perhaps boldly, and economically. The endeavor has been made to formulate projects which are comprehensive, co-ordinated, practical and economic. Revisions and additions may be required to a certain extent. A high standard of engineering is imperative in dealing with flood problems lest failure of works designed to withhold floods result in greater catastrophies than the floods themselves. In the investigation and design of flood-control structures the Corps of Engineers has upon occasion retained many eminent engineers from civil life, as well as eminent geologists, in a consulting capacity. After all the years since the early settlement of the United States, it appears that at last flood control has attained a proper place among the numerous activities of the Federal Government, for the welfare of the country. However, States, political subdivisions thereof, and other responsible local agencies must also do their part.

APPENDIX

Abstract of the Flood Control Act[34] Approved June 22, 1936

An Act

Authorizing the Construction of Certain Public Works on Rivers and Harbors for Flood Control, and for Other Purposes.

Be It Enacted by the Senate and House of Representatives of the United States of America in Congress Assembled,

Declaration of Policy

Section 1. It is hereby recognized that destructive floods upon the rivers of the United States, upsetting orderly processes and causing loss of life and property, including the erosion of lands, and impairing and obstructing navigation, highways, railroads, and other channels of commerce between the States, constitute a menace to national welfare; that it is the sense of Congress that flood control on navigable waters or their tributaries is a proper activity of the Federal Government in cooperation with States, their political subdivisions, and localities thereof; that investigations and improvements of rivers and other waterways, including watersheds thereof, for flood-control purposes are in the interest of the general welfare; that the Federal Government should improve or participate in the improvement of navigable waters or their tributaries, including watersheds thereof, for flood-control purposes if the benefits to whomsoever they may accrue are in excess of the estimated costs, and if the lives and social security of people are otherwise adversely affected.

[34] Public Doc. No. 738, 74th Cong., H. R. 8455.

Section 2. That, hereafter, Federal investigations and improvements of rivers and other waterways for flood control and allied purposes shall be under the jurisdiction of and shall be prosecuted by the War Department under the direction of the Secretary of War and supervision of the Chief of Engineers, and Federal investigations of watersheds and measures for run-off and waterflow retardation and soil erosion prevention on watersheds shall be under the jurisdiction of and shall be prosecuted by the Department of Agriculture under the direction of the Secretary of Agriculture, except as otherwise provided by Act of Congress; and that in their reports upon examinations and surveys, the Secretary of War and the Secretary of Agriculture shall be guided as to flood-control measures by the principles set forth in Section 1 in the determination of the Federal interests involved; Provided, That the foregoing grants of authority shall not interfere with investigations and river improvements incident to reclamation projects that may now be in progress or may be hereafter undertaken by the Bureau of Reclamation of the Interior Department pursuant to any general or specific authorization of law.

Section 3. That hereafter no money appropriated under authority of this Act shall be expended on the construction of any project until States, political subdivisions thereof, or other responsible local agencies have given assurances satisfactory to the Secretary of War that they will (*a*) provide without cost to the United States all lands, easements, and rights-of-way necessary for the construction of the project, except as otherwise provided herein; (*b*) hold and save the United States free from damages due to the construction works; (*c*) maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of War: Provided, That the construction of any dam authorized herein may be undertaken without delay when the dam site has been acquired and the assurances prescribed herein have been furnished, without awaiting the acquisition of the easements and rights-of-way required for the reservoir area: And provided further, That whenever expenditures for lands, easements, and rights-of-way by States, political subdivisions thereof, or responsible local agencies for any individual project or useful part thereof shall have exceeded the present estimated construction cost therefor, the local agency concerned may be reimbursed one-half of its excess expenditures over said estimated construction cost: And provided further, That when benefits of any project or useful part thereof accrue to lands and property outside of the State in which said project or part thereof is located, the Secretary of War with the consent of the State wherein the same are located may acquire the necessary lands, easements, and rights-of-way for said project or part thereof after he has received from the States, political subdivisions thereof, or responsible local agencies benefited the present estimated cost of said lands, easements, and rights-of-way, less one-half the amount by which the estimated cost of those lands, easements, and rights-of-way exceeds the estimated construction cost corresponding thereto: And provided further, That the Secretary of War shall determine the proportion of the present estimated cost of said lands, easements, and

rights-of-way that each State, political subdivision thereof, or responsible local agency should contribute in consideration for the benefits to be received by such agencies: *And provided further*, That whenever not less than 75 per centum of the benefits as estimated by the Secretary of War of any project or useful part thereof accrue to lands and property outside of the State in which said project or part thereof is located, provision (*c*) of this section shall not apply thereto; nothing herein shall impair or abridge the powers now existing in the Department of War with respect to navigable streams: *And provided further*, That nothing herein shall be construed to interfere with the completion of any reservoir or flood control work authorized by the Congress and now under way.

Section 4. That consent of Congress is hereby given to any two or more States to enter into compacts or agreements in connection with any project or operation authorized by this Act for flood control or the prevention of damage to life or property by reason of floods upon any stream or streams and their tributaries which lie in two or more such States, for the purpose of providing, in such manner and such proportion as may be agreed upon by such States and approved by the Secretary of War, funds for construction and maintenance, for the payment of damages, and for the purchase of rights-of-way, lands, and easements in connection with such project or operation. No such compact or agreement shall become effective without the further consent or ratification of Congress, except a compact or agreement which provides that all money to be expended pursuant thereto and all work to be performed thereunder shall be expended and performed by the Department of War, with the exception of such reasonable sums as may be reserved by the States entering into the compact or agreement for the purpose of collecting taxes and maintaining the necessary State organizations for carrying out the compact or agreement.

Flood Control Act of 1936

Section 5. That pursuant to the policy outlined in Sections 1 and 3, the following works of improvement, for the benefit of navigation and the control of destructive flood waters and other purposes, are hereby adopted and authorized to be prosecuted, in order of their emergency as may be designated by the President, under the direction of the Secretary of War and supervision of the Chief of Engineers in accordance with the plans in the respective reports and records hereinafter designated: *Provided*, That penstocks or other similar facilities, adapted to possible future use in the development of adequate electric power may be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

FLOOD PROTECTION DATA[1]

PROGRESS REPORT OF THE COMMITTEE

A marked interest was evident in 1936, in the collection and publication of flood data. The attitude of the Federal Government in providing funds for this purpose has been most gratifying. The report on Deficiencies in Basic Hydrologic Data submitted by the National Resources Committee, in September, 1936, is helpful as it sets forth clearly the Committee's earnest belief that there is necessity for an expanded hydrologic-data program.

Among the many flood studies undertaken during 1936 should be mentioned those made by the Water Resources Branch of the United States Geological Survey, which are expected to be published in its *Water Supply Paper* series for general distribution. A report, entitled "The New York State Flood of July, 1935", has already appeared in the form of *Water Supply Paper No. 773-E.* It contains information on the rainfall conditions which caused the flood and gives flood discharges at many points on medium as well as small-sized streams. A report on the Republican and Kansas Rivers flood of 1936 is in an advanced stage of preparation; it likewise covers both rainfall and storm run-off. A report on the flood of January 1, 1934, in La Canada Valley near Los Angeles, Calif., is also in preparation. In addition to rainfall and run-off conditions incident to this flood, this report will be of particular interest in giving data concerning the extraordinary débris movement which took place during the flood. A study of the cloudburst floods which occurred in Texas in 1935 is under way. It should contribute measurably to the scant information available on storm run-off from high rates of rainfall peculiar to that section of the United States. A survey and report are in progress, under an allocation of funds by the Public Works Administration, on the floods of March, 1936, extending from Maine to Virginia and westward to the Ohio River Valley. An allotment of PWA funds will enable the Water Resources Branch to make an investigation of stages and discharges for the record-breaking floods which occurred in Texas in September, 1936.

The Corps of Engineers, U. S. Army, reports having made the following investigations of flood problems, reports on which were submitted to Congress during 1936: Red River and Tributaries, Louisiana, Arkansas, Oklahoma, and Texas (published in H. Doc. 378, 74th Cong., 2d Sess.); Yakima River, Washington (published in H. Doc. 399, 74th Cong., 2d Sess.); Connecticut River, Connecticut, Massachusetts, New Hampshire, and Vermont (published

[1] Presented at the Annual Meeting, New York, N. Y., January 20, 1937.

in H. Doc. 412, 74th Cong., 2d Sess.); Point Remove Creek, Arkansas (published in H. Doc. 450, 74th Cong., 2d Sess.); also the following as yet unpublished reports; Salt River, Missouri; Gafford Creek, Arkansas; Sebawaing River, Huron County, Michigan; Oswego, Oneida, Seneca, and Clyde Rivers, New York; Snohomish, Skyhomish, and Snoqualmie Rivers, Washington; Purgatoire and Apishapa Rivers, Colorado; and Goldsborough Creek, Washington. In addition, the Corps of Engineers was directed by the last Congress to undertake investigations on more than two hundred streams in various parts of the United States, as specified in the Flood Control Act of June 22, 1936. Under the Overton Act of June 15, 1936, provision is made for the completion of the program of flood control in the Lower Mississippi Valley.

Perhaps one of the most important studies relating to rainfall and run-off, with special reference to floods and flood control, is that being conducted by the Tennessee Valley Authority. Under its direction and in co-operation with the United States Weather Bureau and the U. S. Geological Survey, many rainfall and river gages, mostly of the automatic type, have been installed at controlling points in the water-shed. The studies so launched are the most detailed of their kind undertaken on a large water-shed (more than 40 000 sq. miles) in the United States.

Special interest attaches to the investigations into the effect of water-shed protection on storm run-off being made by the Soil Conservation Service of the U. S. Department of Agriculture. These studies are aimed at bringing about a clearer understanding of the relation between head-water storm run-off and floods in rivers. This is a much neglected chapter in flood-protection data and one which, properly developed, may exert a profound bearing on future planning of flood control. Much careful investigation still is required before available data can be accepted as safe criteria on which to predicate important projects.

Deserving of special mention as valuable contributions on the subject of historic floods and of flood magnitudes and frequencies are the papers published by the Boston Society of Civil Engineers[2] in the form of a Symposium on the March, 1936, Floods in New England. Among the contributors are: Arthur T. Safford, and Richard S. Holmgren, Members, Am. Soc. C. E.; and H. B. Kinnison, and Harry M. Nelson, Assoc. Members, M. Soc. C. E.

A work of outstanding value is the thesis by S. A. Weakley, M. Am. Soc. C. E., on "Floods in the Cumberland River"[3], which carries the record at Nashville, Tenn., back to early historic days.

Space does not permit mentioning the numerous special studies which have been conducted during the year, nor the many papers and discussions that have been published in connection therewith, all relating to storm rainfall and flood run-off in various parts of the United States. Considered collectively they are significant of the deep interest that is being manifested

[2] *Journal*, Boston Soc. of Civ. Engrs., October, 1936.

[3] Presented to Vanderbilt Univ., in partial fulfillment of the requirements for the degree of Civil Engineer.

by the Engineering Profession in flood data and the need that is felt for disseminating this class of information.

The aggregate mass of flood data made available as the result of all these studies represents a material contribution to the knowledge of floods in the United States. However, before the data can become of permanent practical value to engineers, it will require a special agency to collate, tabulate, and publish them in convenient form for reference.

A paper on "Flood Stage Records of the River Nile", by C. S. Jarvis, M. Am. Soc. C. E.[4] should be mentioned. Although it is not concerned with floods in the United States, it serves to shed important light on Nile River floods over a period covering more than 1 300 yr, the longest record of its kind in existence.

That much still remains to be done toward collecting and publishing flood data is amply demonstrated by the flood events of 1935 and 1936. It is perhaps fortunate that many new records in both flood stages and discharges should have been established on large as well as on small streams. These records are proving of greatest assistance in confirming the expectations regarding high rates of run-off that must be provided for in flood-control works, and which previously had been matters of estimate on which opinions have differed. As flood damage in 1935 and 1936 reached extraordinary figures, demand for better flood-forecasting service has been stressed repeatedly. This in turn, emphasizes the need for more and better field observations on rainfall, water-shed conditions, and river stages. Had it been known, for instance, in March, 1936, that most of the water-sheds affected by storm rainfall and snow melting had deeply frozen soil beneath the thin veneer of thawed top-soil, better forecasts might have resulted. Apparently, it was nobody's business to furnish data on such an important detail.

There appears to be need for a systematic campaign to educate the public on the dangers of locating industries, municipal pumping plants, electric power stations, hospitals, and penitentiaries in the natural paths of flood waters; also, as to the need of enlisting the services of experts on flood run-off whenever indispensable structures, such as bridges, dams, highways, or railroads, must be built across streams or in their flood-plains. Unless the lessons taught by flood data are heeded, the United States will witness the same difficulties that have beset foreign countries. Cities such as Rome, Italy, Paris, France, London, England, Cologne, Germany, and Moscow, U. S. S. R., have labored intermittently for centuries on their respective flood problems but have not to this day succeeded in conquering them. The same is true of many smaller cities in Europe. In the United States there are towns situated on islands and on low alluvial ground where no towns should ever have been built; yet in most of these cases flood records extending over a century or more afford ample proof of the frequencies with which disastrous floods must be anticipated. Despite such factual data, cities thus situated continue to grow and are daily adding to their flood hazards instead of reducing them. This situation, in large part, is attributable to ignorance and,

[4] *Transactions* Am. Soc. C. E. Vol. 101 (1936), p. 1012.

in part (especially of late), to unlimited faith in the possibilities of flood control. Although this Committee recognizes the futility of attempting to educate the general public, it feels that much might be accomplished toward preventing capital from establishing industries and creating towns on low ground subject to frequent flooding, through the medium of well-devised publicity which would quote facts concerning floods and flood damage. It is useful in this connection to refer to the long-established fact that it is not flood flow that is increasing in the United States, but that flood damage is increasing due to the ever-increasing amount of damageable property that is being placed within the reach of flood waters. This Committee is of the opinion that suitable publicity will check the growth of this unwarranted economic waste.

What makes the flood problem, broadly speaking, so little understood even among the educated, is that so much of the knowledge relating to storm rainfall and run-off gained in any particular locality affords no reliable index as to what may be expected elsewhere. The main obstacles that have hindered progress in the proper interpretation of flood data are: (1) The lack of sufficient data; and (2) the human tendency to generalize and to overlook the fact that no two water-sheds are alike, and that each flood problem must be treated as being in a class by itself. These are age-old maxims as applied to rivers generally; but now they are coming in for increased attention in connection with the newer methods of water-shed protection calculated to reduce storm run-off. The fact that an acre of forest-covered land on an experimental farm showed less than 2% run-off after a heavy shower is no indication as to what percentage of run-off it might yield at the end of a 5-day continuous storm such as occurred in 1913 over the Ohio Valley, but which the experimental farm has not yet experienced; nor is the infiltration on any experimental plot any measure of what may be expected from a large water-shed in which reforestation and soil conservation, however scientifically applied, could not hope to cover more than 50% of the land surface, assuming a population of average density living in the water-shed. Much remains to be determined as to the true degree of effectiveness of various methods of water-shed protection during great storms lasting four or five days, such as have been responsible for some of the major floods in the United States.

Out of all that has been said and written of late regarding floods, there emerges one point on which there is general agreement, namely, that great floods have occurred as far back as historical data are available, even in the days of the primeval forest and before the buffalo grass on the great plains had been removed, and that great floods will continue to recur in the future. Even the most ardent exponents of head-water protection and a storm run-off retardation by means of soil conservation, reforestation, cover crops, and small reservoirs, are agreed upon this point.[5]

This Committee was appointed in 1934 primarily to serve in an advisory capacity to assist the engineers of the U. S. Geological Survey with the

[5] *Soil Conservation*, July, 1936, p. 6, and November, 1936, p. 91.

compilation and publication of flood data. This assignment having been fulfilled early in 1936 with the publication of the data in *Water Supply Paper 771*, entitled: "Floods in the United States—Magnitude and Frequency", no objectives have been before the Committee other than the general theme of promoting research and encouraging a systematic compilation of flood data generally. Consideration has been given to the question as to what constitutes a logical repository for such data. The consensus of opinions within the Committee, as well as among those members of the profession contacted, is: (1) That the proper agency to handle the collection and publication of flood data should be a *permanent* Federal, rather than a private agency; and (2) that, whatever Federal Bureau may be designated for the task, it should be furnished with adequate funds and specially trained personnel. The thought is that, however complete may be the stock of data in its custody, such a Bureau should be capable of furnishing reliable basic information upon request; and have experts fully capable of making sure that the data catalogued are reliably accurate for engineering purposes, and what data, if any, must be considered by the user as approximate.

This Committee desires to reiterate its position with regard to the recommendations submitted by it in its Annual Progress Report[6] for 1935, more especially as regards urging the Federal Government to:

(*a*) Continue the collection and publication of flood data in tabulated form, to include all records of importance which have not yet been published in public document form;

(*b*) Make an inventory of the great historic floods of early days; and,

(*c*) Make a study of cloudburst floods, their causes and frequency of occurrence.

This Committee further recommends that the Board of Direction urge upon the proper Federal officers:

(1) That future publication of flood data by the Federal Government be in pamphlet rather than in book form, the contents of each pamphlet to be restricted to one river basin or geographical group of streams, for the purpose of facilitating prompt publication and early distribution as well as frequent re-publication in order to keep printed records up to date;

(2) To initiate a campaign of education aimed at checking the ever-growing annual toll of damage by floods. The method suggested is by means of publicity pointing out the dangers of locating valuable property in flood zones. This is putting available flood records to practical use for the common good.

This Committee senses the limitation in the scope of its activities as imposed by its title. There is need for the Society to give consideration to important aspects of flood-control engineering which are confronting the profession but which are not within the purview of the Committee on Flood-Protection Data.

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[6] *Proceedings*, Am. Soc. C. E., February, 1936, p. 203.

The Committee, therefore, endorses the authorization and appointment by the Board of Direction of a committee to make a study of the various methods of controlling floods, with particular reference to their physical and economic limitations.

Respectfuly submitted,

GERARD H. MATTHES, *Chairman*,
FREDERICK H. FOWLER,
ROBERT E. HORTON,
IVAN E. HOUK,
CHARLES W. KUTZ,
CHARLES W. SHERMAN,
DANIEL C. WALSER,

December 6, 1936. Committee on Flood Protection Data.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

VARIED FLOW IN OPEN CHANNELS OF ADVERSE SLOPE

Discussion

BY ARTHUR E. MATZKE, JUN. AM. SOC. C. E.

ARTHUR E. MATZKE,[28] JUN. AM. SOC. C. E. (by letter).[28a]—The case of varied flow in open channels of adverse slope occurs rather seldom in practice. The extent of the discussion, therefore, was rather surprising and most pleasing. The rarity of occurrence also made the fact that the method presented in the paper was applied in such an important structure as the Cape Cod Canal even more gratifying. The kind reference made by Captain E. C. Harwood[29], Corps of Engineers, U. S. Army, would seem to offer ample compensation for the time spent.

The approach of most of the discussers was from a rather general point of view without direct reference to any particular practical application. In fact, much of the discussion extended beyond the scope of the problem actually treated and compared methods to be used in handling general cases of varied flow. Messrs. Stevens and von Bergen centered their remarks on the relative merits of computations by means of the varied flow function by comparison with the method of successive approximations.

Historically, the method of successive approximations begins with the work of Belanger (1827). In fact, his treatment is so complete that, fundamentally, nothing has been added since. It is interesting to note, however, that the tendency and the endeavor in the past hundred years on the part of Belanger's pupils and legatees was to substitute for the treatment by approximations a more analytical solution obtained by integrating the varied flow equation. The work of Dupuit (1848), Bresse (1860), Rühlmann (1880), Tolkmitt (1892), Schaffernak (1914), and Professor Bakhmeteff (1932), might be viewed as the gradual development of this idea. Indeed, Professor Mavis

NOTE.—The paper by Arthur E. Matzke, Jun. Am. Soc. C. E., was published in February, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1936, by Messrs. H. E. von Bergen, W. E. Howland, and Arno T. Lenz; and August, 1936, by Messrs. J. C. Stevens, F. T. Mavis, and Hunter Rouse and Merit P. White.

[28] Research Asst., Dept. of Civ. Eng., Columbia Univ., New York, N. Y.

[28a] Received by the Secretary January 25, 1937.

[29] "Proposed Improvement of the Cape Cod Canal", *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1473.

seems to characterize this application of integral calculus as nothing more than a convenient method of summation of a column of figures that represent some regular, analytical (or graphical) array of numbers.

There is another point which should be mentioned in the hope of clearing up a misconception. The comparison of methods by Messrs, Stevens and Mavis is presented principally with regard to natural watercourses. As far as the paper is concerned, natural watercourses were not mentioned. In fact, Professor Bakhmeteff specifically mentions the method of successive approximations as particularly applicable to computing back-water curves in rivers. It may be interesting to remember that, in the past, back-water curves in rivers constituted the principal practical subject in the realm of varied flow. However, at present, the complex cases arising from irrigation, water power, and navigation projects, with canals working under different levels and discharges, have practically transferred the problems connected with them into a different field. As an example, one may cite the delivery problem (Example 3). Any one who would try to solve it by approximations instead of the varied flow function would soon realize the actual state of affairs.

The writer is grateful to Messrs. Stevens and Howland for demonstrating the inaccuracy in part of the tables of values of the varied flow function. The simple graphical method which was used in obtaining these tables is presented in the paper. This was done to enable those who found the tables inadequate for their purposes to obtain satisfactory values. Since the method was graphical, the limit of accuracy of the values is a function of the scale of the drawing.

In deriving Equation (21), which is described as "perfectly general", Mr. Stevens refers to Fig. 7(a) which depicts a sustaining slope. He then writes a formula (Equation (20)) in which $\Delta\epsilon$ is defined as "the loss of specific energy in the reach, Δx." Consideration of the specific energy curve and the fact of the existence of surface curves of the type referred to by Professor Bakhmeteff as S_1, S_2, S_3, and M_3 shows conclusively that it is possible for $\Delta\epsilon$ to be a gain in specific energy as well as a loss. Wherever the work done by gravity in transporting a quantity of water over the distance, Δx, is greater than the work done by friction, etc., in the same process, it follows necessarily that the difference between the two must be stored in the water in the form of an increase of specific energy. This is fundamental and highly important. In Fig. 1 and throughout the paper, S_0 is a symbol for bottom slope just as it is in Mr. Stevens' discussion, and does not refer to a "normal slope."[29a]

Thanks are due Mr. Lenz for his investigation and verification of the exponential relation between K and y. The regular behavior of his discharge rating curve is most convincing. It answers in part, at least, the doubts concerning the behavior of n expressed in other discussions. The writer is also deeply grateful to Messrs. Rouse and White for their truly rigorous development and generalization of the solution.

[29a] Correction for *Transactions:* Change "s" and "s_0" to "S" and "S_0" throughout the paper.

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DISCUSSIONS

ADMINISTRATIVE CONTROL OF UNDERGROUND WATER: PHYSICAL AND LEGAL ASPECTS

Discussion

By Messrs. John E. Field, George S. Knapp, and Chandler Davis.

John E. Field,[43] M. Am. Soc. C. E. (by letter).[43a]—Dealing as it does, with a very complicated subject, this paper is unusually comprehensive and clear. For simplicity, it would be well to separate the discussion into two major parts: First, where riparian rights obtain—geographically east of the 98th Meridian; and, second, where the appropriative doctrine rules (that is, west of that line). The westerly area should be subdivided into the several States, and the one which offers the least departure from the strictly appropriative doctrine and in which, through judicial decision, the rules are becoming fixed, should be discussed first. With a code for such a State as a basis, modifications in conformity with laws and customs can be made and applied to other States.

Colorado seems to be the State where riparian rights intrude the least, and where the laws and practice have reached a fair degree of stability. The practice in Colorado is based largely on judicial decisions and many of the statutory laws have followed rather than preceded the decisions of the Court. Until 1879, the practice was "go as you please" with only the Constitution as a guide, which provided that "the right to divert water for beneficial purposes shall never be denied" and "the first in time is the first in right." In 1879, a law was passed providing for "proving up" on claims of the irrigator, of recording and defining the rights, and for "adjudication." State-wide adjudications were had in the period from 1883 to 1886. The

Note.—The paper by Harold Conkling, M. Am. Soc. C. E., was published in April, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: August, 1936, by Messrs. Joseph Jacobs, and W. D. Faucette and J. E. Willoughby; September, 1936, by R. E. Savage, Assoc. M. Am. Soc. C. E.; December, 1936, by Messrs. H. J. F. Gourley and O. J. Baldwin; January, 1937, by Messrs. G. E. P. Smith, and David G. Thompson; and February, 1937, by Wells A Hutchins, Esq.

[43] Cons. Engr., Denver, Colo.

[43a] Received by the Secretary January 15, 1937.

right to transfer water from one canal to another was recognized and although it sometimes led to injustice to the individual, nevertheless, it encouraged economy of use and caused the transfer of water from the poorer land to the more productive. These transfers were decidedly in the public interest, and, in part, they account for the rapid development in Colorado. On the other hand, State governments that tried to tie the water to the land and to avoid individual injustices, lagged in their development. A statute prescribing the method of effecting the transfer was passed in 1899, which the State Supreme Court declared was the expression of the existing law and merely made definite the procedure to be followed.

In the matter of laws, regulations, and administration of underground waters, as the author indicates (see "Underground Water Law in General: Colorado"), the Colorado Courts have applied the same regulations as for surface water, but the time has now arrived when the method of administering and recording underground water rights should be embodied in statutory law, based on practice, custom, and the decisions of the Courts. This orderly procedure which has characterized the evolution in water laws in Colorado, will result in an equitable and practical expression of pre-existing fact. It is for these reasons that the writer uses Colorado as the example to clarify his theories and arguments.

As it passes from the mountains to the plains, the South Platte River derives less than one-half its supply from the South Park area (see "Underflow Which May Support a Surface Stream.") It passes through Denver about ten miles east of the foot-hills and at the bend near Greeley, is 30 miles from the mountains and enters Nebraska close to the northeast corner of Colorado. The minimum precipitation on the eastern slope (about 12 in.) is found about 50 miles east of the mountains; it increases to the west to 15 in. at the foot-hills, and increases easterly to about 16 in. at the Kansas-Colorado line. Due to the rapid fall in the river (about 2 000 ft in 200 miles), existing canals could be extended to cover the greater portion of the area east of the mountains and between the Wyoming line on the north to the South Platte-Arkansas divide on the south, and water could be carried on to land in the Arkansas water-shed or, following the divide, enter Kansas at the point of highest elevation in that State. In other words, practical gravity canals can reach lands far in excess of the water supply and if the present supply could be increased (through trans-mountain diversions) by 50%, only moderate extensions of existing canals would be required to use the increase. This possibility explains the absence of pumping. Pumping to provide storage space has not been proposed heretofore, for the South Platte Valley, as it is water and not reservoir space that is needed; and, furthermore, diversion dams now raise the underflow into many canals quite as effectively as pumps could. Below these dams, during the period of ordinary flow, the river bed is dry and would remain dry for the remainder of its course, except for the invisible inflow of natural and return seepage water, which amounts to about 1 000 cu ft per sec, or 5 cu ft per sec per mile of river.

Consequently, the writer must differ with Mr. Conkling in his facts and conclusions as stated under the headings, "Underflow Which May

Support a Surface Stream" and "Administration: Underground Water", as far as they refer to conditions on the South Platte.

Numerous pumping plants on the mesa lands are used to supplement the canal supply. These wells are below the canals and their supply is from seepage water on its way, underground, to the watercourses. The right to pump has not been seriously questioned due in part to the difficulty of proving that the pumped water would reach the stream at a time when needed by a plaintiff. The damage in reality is inconsequential to any one canal. One-half the seepage water reaches the river in the non-irrigation season and the other half is diverted and demanded by one canal to-day and another to-morrow. All are probably affected at one time or another, each in its turn, depending on the stage of the river flow and the dates of priority. If all factors were determinable, canals that are remote and in other districts, may be the ones damaged at some particular time. A small quantity of the return seepage is intercepted by the pumps, but which canal is damaged, and to what extent, is impossible of even approximate determination. The only benefit that pumping would accomplish would be to furnish, from the underflow, some water to meet the emergency needs of canals. Numerous pumps have been installed in the five years (1932 to 1937), for such emergency purposes and the "live and let live" attitude of canal administrators who have been adversely affected has permitted their use, because each canal administrator suspects that he too will sometime wish to do the same thing. Contrary to the author's contention, the South Platte does not offer an example of the wisdom or possibilities of pumping; nor will pumping create reservoir capacity that may be filled by water now escaping.

Under "Artesian Areas or Basins", the author does not mention the great artesian wells of the San Luis Valley, near the headquarters of the Rio Grande. Schlicter gave their number as about 6 000, and the aggregate supply as about 300 000 acre-ft. After more than forty years, they are still active and such as show a decreased discharge have probably become choked, the casing has failed, or the water is escaping before reaching the surface.

Table 1 seems to be in serious error in the columns, "Areas Supplied from Streams with Supplemental Well Supply", for in the San Luis Valley there is an area of more than 200 000 acres receiving water from wells, in addition to their canal supply. The area is sub-irrigated, and so this item of double source of supply escaped the eye of the field agent taking the census. In Table 2, remarks as to Colorado should read: "Pumping for emergency purposes only, is advisable." An increase in water supply will be necessary before much further development is possible. Otherwise, only through economy of use, can any further development of consequences be expected east of the mountains in Colorado. The error mentioned lays the accuracy of Table 1 open to suspicion. In any case the data pertaining to agriculture in 1919, are eccentric (that is, out of balance), especially when considering trends. They should be used with great caution as the stimulus to agriculture due to the World War makes them unreliable.

The question, "Is the diverter from a stream protected in its means of diversion?" (see "Water Laws in General") is answered by a decided "No". It is quite enough that the early appropriator enjoys at all times an exclusive 100% right (when it is in the river) and, although he is entitled to the maintenance of conditions as of the date of his appropriations, to have it apply to the means of diversion would lead to intolerable conditions, contrary to the public interest. For example, when, by reason of diversion of the water above, the first appropriator finds his head-gate too high and the water too low for him to divert his full right, it would be wasteful to send sufficient water down to him to raise its surface to the old gage height; and, therefore, the law provides that he may move his head-gate up stream, build a diversion dam, or put in a pump. Assuming that it is Canal No. 2 which causes the low flow, to compel Canal No. 2 to pay for the change might well impose such a burden on that canal as to prevent its construction. Canal No. 1 could, by reason of its preference right, better afford to assume the burden. The author states that (see "Water Law in General") "the hurdle, faced by the individual who proposes to divert at an up-stream point would indeed be high, if he had to compensate all down-stream appropriators." Although there are no statutes in Colorado which protect the appropriator in his means of diversion there are statutes which permit him to take such measures as will enable him to secure his full appropriations and which by implication denies him the right to insist on a *status quo* as to his means or instrumentality of diversion.

In the case of overflow on meadows, when the river falls so low by reason of diversions above, that the meadows are no longer overflowed, the law permits the construction of canals, with a date of priority as of the date of its first use as a meadow.

In a recent decision of the Supreme Court of Colorado, the Court reversed itself and ruled that reservoirs and canals were on the same basis—and that the first in time, was the first in right. Here, it is quite definitely indicated that the instrumentality employed to accomplish beneficial use, in no way affects the right of use. Therefore, pumps have no different standing from gravity canals, reservoirs, pipe lines, or carrying the water in a bucket. Such being the case, it seems that in Colorado a practical plan of administration alone remains to be evolved and that at this time, the Administrator, the State Engineer, and Water Commissioner, should be given definite rules and clear instructions under which they will perform their administrative duties in handling underground water. The statute should require that filings be made as in the case of canals, that all wells to be used, for other than household and stock purposes, must be drilled under the direction of the State Geologist who would preserve a careful log of the well and make a study and report on the geology, the replenishment, and the source of supply. This report would be filed with the State Engineer, who, in turn, would make a tentative determination of the duty of water, and at the beginning and end of each irrigation season would check the height of the water-table. The statute should require the installation of a meter, a reading of which would be taken by the Water Commissioner as often as the State Engineer deemed

advisable, or the latter could require the installation of a self-recording meter. The quantity of water pumped would be limited by the tentative duty of water as fixed by the State Engineer and the acreage irrigated, and the quantity pumped each year would be recorded and would form the basis for later adjudication.

Adjudication should be initiated by the State Engineer when, in his judgment, it was advisable, or upon the petition of the water users. In this adjudication, the Court would fix the depth below which the water could not be drawn, based on the records gathered and compiled by the State Engineer or by any competent investigator. The appropriator should be limited in the quantity of water he could pump in any one year; but the time or the rate of flow should be at the discretion of the user. The State Administrators would be concerned principally in the gross acre-feet pumped per year. The tentative duty of water would be changed from time to time until a fixed duty could be determined on the basis of the records. At first, the tentative duty might well be placed at 1.25 acre-ft per yr, which approximates the consumptive use for Colorado, a use over and above the rainfall. After the adjudications were had, the water officials would act thereunder, shut down such pumps as had reached the quantity allowed by the decree, or when the water-table approached its minimum allowable elevation. In periods of extra large supply, or of drought, the pumps with the later decrees would be allowed to run or would be closed down. After a period of years, the replenishment possibilities would be determined quite closely and the junior decrees would, in practice, become void. The writer does not anticipate that there would be an under-development; if this should occur, there would be almost no call for administrative control.

If pumps should be installed in the bottom-lands (land a few feet above the stream), they would be allowed to operate in the order of their priority; that is, after the demands of prior rights had been satisfied. Pumps more remote from the river should be allowed to operate until holders of earlier priorities make complaint, in which case, the State Engineer would investigate and render his decision, which would be binding until altered by the Courts.

In time, the necessity of classifying the land would probably arise, the entire underground supply in some areas would be reserved for domestic and stock uses, and, in others (as in river bottoms) all water pumped would be considered as a diversion from the river supply. In the matter of preference uses, the laws now provide that domestic use is superior to agricultural use, but by the Court's decision this means that the water used for irrigation may be condemned and converted to domestic use.

Summary, Discussion, and Conclusions.—In addition to what the author states in Item 2 of his "Conclusions", it should be added, that the damage occasioned by the decrease of supply on the South Platte is inconsequential, as it is shared by all priorities, each in its turn. Referring to Item 4, the fundamentals of Colorado laws were adopted before Statehood (in fact, coincident with the creation of the Territory in 1861). The same is true of most of the other arid States.

The practical operation of Colorado laws and decisions relative to underground water (see Item 5), may have prevented the development of pumping to some extent, but they have prevented infringement on existing rights, have protected developed areas, and have prevented unwise and "wild-cat" enterprises. The writer, with an intimate knowledge of the State for more than fifty years, is unable to visualize a materially greater development by the use of pumps than has occurred without them; and, on the other hand, he can readily visualize interminable legal controversies and a less development had the California law prevailed. In fact, the physical conditions in Colorado and in California in the matter of underground waters are so different that no comparison can be made; nor can conclusions be drawn with profit.

Much research and investigation are necessary for accurate and intelligent control (see Item 19), but unless the development is sudden and complete, the data can be gathered over a period of years, and as a part of the ordinary administration duties. The cost will be borne largely by the appropriator, just as it is now borne by canals in the construction of weirs, registers, and co-operative measurements. The administrative control and the gathering of data should not wait until overdraft threatens, but to be of greatest value, must be gathered during the years of development, and must be available when overdraft threatens and adjudications are asked.

In conclusion, the paramount consideration in such matters as the paper touches, is the public interest; the greatest economical development possible, even at the sacrifice or infringement of individual or property rights, should be the objective. Old laws must be abrogated and new ones enacted to meet physical and economic conditions. In the arid States, the riparian doctrine was ruthlessly set aside, or so modified as to meet the necessities of the people. Most of the early Court decisions were based on the necessities, and on common sense rather than on precedent, or the common law; and it is to be hoped that the Courts will continue to recognize paramount necessities as controlling, rather than adhere to the outworn laws and outworn decisions.

The State of Colorado has been very fortunate in having a Court willing to reverse itself and willing to consider facts and the public interest in making its decisions; and, in Wyoming, the Court, recognizing the benefits of the trànsfer of water, "hedged" on the "water-attached-to-land" constitutional provision and held that this provision did not apply to ditches built before the Territory became a State. It is axiomatic that water should be used where the user can pay the most for it.

In a mining camp, in Colorado, a "district" was formed, about 1859, and rules and regulations were adopted. In conclusion this document reads: "In waters not covered by the above rules, the laws of the United States shall apply." The writer's theory also is, that "when existing laws and decisions do not conflict with the public interest, they shall prevail."

GEORGE S. KNAPP,[44] M. AM. SOC. C. E. (by letter).[44a]—Studies such as those made by Mr. Conkling in the preparation of this paper should aid

[44] Chf. Engr., Div. of Water Resources, Kansas State Board of Agri., Topeka, Kans.

[44a] Received by the Secretary January 26, 1937.

materially in an understanding of ground-water administrative problems, and in a region as young, relatively, as that of the Western States, where ground-water laws are still in a formative state, the paper should be a valuable guide in the shaping of legislation for administrative control.

The paper shows the great diversity among the various States in laws governing rights to the use of ground-water. In their scope they vary from those vesting absolute and unrestricted ownership to ground-water in the ownership of the land under which it is found, to those which apply the appropriative doctrine so strictly that, in some States, a land owner may be barred from using any of the water under his land for irrigation because owners of other land have been permitted to appropriate the entire supply.

The object of such laws and administrative regulations is to permit the orderly development of ground-water supplies and, at the same time, to prevent over-development of the supply and injury to water users. Some effort has been made in recent years to secure the enactment of uniform ground-water laws for irrigation in the Western States. Uniformity of State laws is desirable in so far as it can be attained. With respect to ground-water under land no longer in public ownership, the process of changing the law is difficult, particularly if it is attempted to change from some form of the riparian or ownership doctrine to the appropriative doctrine. The process of separating the ownership of the water from that of the land by legislative enactment would undoubtedly be held by the Courts as the taking of property without due process of law; that is, without condemnation and compensation.

In some States both surface water and ground-water are under the same rule of law. In others, it is not. A number of States attempt to apply the rule of priority of right to ground-water in the same manner as it is applied to surface water. In this connection, it should be noted, perhaps, that the appropriative doctrine does not apply to ground-water, as among various appropriators, with the same effect, and in the same manner, as it does to surface streams. The available quantity of ground-water does not fluctuate rapidly from day to day as surface water does. Even if it moves in well-defined channels, ground-water constitutes essentially a reservoir and not a stream. Its surface is drawn down slowly with use, and the supply is replenished slowly. It is only the earliest appropriators from surface streams who enjoy their full measure of water in all years. The remainder receive their appropriation on a fewer number of days, depending on the relative priority of their appropriations. The last appropriators may be entitled to water only during the few days from time to time when the stream may be at flood-stage. In applying the appropriative doctrine to ground-water it is necessary to determine the quantity which may be withdrawn each year, and from year to year, without depletion of the reservoir. That group of users, whose aggregate appropriations come within this quantity, becomes, together, a preferred group which receives a full supply each year, whereas the latter would-be appropriators, are denied any part of the water supply.

There has been a great increase in the acreage irrigated from wells in the Arkansas and Platte River Valleys in Colorado, Kansas, and Nebraska, since

the 1929 census data were secured. Much of it has occurred during the five years of drought experienced in this region in 1932–37. Some of these recently constructed pumping plants are used for the irrigation of new land. Others are used as a supplemental supply for land hitherto "dry-farmed", or for land under canal systems, either pumping into the canals, or directly on to the land to supplement deficient surface supplies. All these pumping plants are situated in the river valleys. As Mr. Conkling states, no extensive use of this ground-water can be made without diminishing the flow of the streams to users below. If the rule of priority of right had been enforced in Colorado, these pumping plants, being junior to canals down stream, as to dates of construction, would not have been permitted to operate when the lower canals were short of water. The immediate effect of the operation of these pumping plants would be the lowering of the water-table in the valley rather than a reduction of stream flow, since the water-table would necessarily have to be lowered before stream flow could be affected. At the time, therefore, they virtually drew on a reserve water supply during a period of drought. To permit them to continue to operate regularly from year to year, however, will result in an increased irrigated acreage, or an increased use of water at such points to the injury of surface-water users down stream.

Ground-water supplies constitute a reservoir that can be held in reserve for long periods without the evaporation losses sustained in the storage of surface supplies. The value of these underground supplies in supplementing deficient stream flow during the recent drought suggests the possible wisdom, as a matter of public policy, of holding them in reserve in certain localities to be used only in times of drought, but that could not be done legally under either the appropriative or the absolute ownership doctrines. It would require an entirely new rule of law.

Ground-water laws are still in the making throughout the West. As they develop it scarcely seems probable that they will grow in uniformity among the States as much as they will grow to meet the different needs and the different conditions of use in the various States.

CHANDLER DAVIS,[45] M. AM. SOC. C. E. (by letter).[45a]—The engineer's mission is to establish the mean number of gallons of water that may safely be pumped from the underground reservoir; the exploitation of the source should be controlled by the Federal Government or by the State Government, and the pumping should be limited to the quantity determined by the engineer; thus, there need be no fear of overdrawing on the sources of supply and causing alarm as Mr. Conkling states happened in California, in areas exploited prior to the organization of the State Division of Water Resources.

Mr. Conkling is correct in advocating that the exploitation of an underground water source be preceded by a thorough investigation, that pumping stations be erected only under license and supervision of some branch of Government, and that the pumping be limited to the findings of the engineer.

[45] Cons. Engr., New York, N. Y.

[45a] Received by the Secretary January 19, 1937.

To emphasize these views, the writer wishes to comment briefly on the status of underground storage in Long Island.

After the recession of the ice of the last glacial period, what is known to-day under the name of Long Island, New York, covered a much larger area than it does now, the south shore extending more than 100 miles farther out to sea. When the subsidence occurred the clay deposited by the melting ice settled below the surface of the Atlantic and is found at various depths, and in various positions (horizontal layers, vertical, and others at all angles). Soon afterward, the sea began to pile sand on the shore front eventually forming the present Island, with its highest dune or backbone nearer the north shore, so that the slope to the north is quite steep, whereas the land slopes gently to the south. Similarly, the ground-water slopes toward the Atlantic very gradually, the highest point of the water-table being south of the highest point of the land. Streams and ponds are found where land contours cut the water-table, their size varying with the rainfall; that is, they shrink after a period of drought and grow larger after a very heavy rainfall period. A perceptible lag is observed due to time required for the rain (which is absorbed by the ground) to reach the water-table. There is one interesting exception, the high lying Lake Ronkonkoma, which does not seem to have any connection with the remainder of the underground water of the Island and seems to follow rules of its own.

The Borough of Brooklyn received its water supply from a line of wells driven along the south shore of the Island. Between 1900 and 1905, a group of farmers sued the City for diverting the water from beneath the farms situated along the line of, and in the vicinity of, the aqueduct. At first the City was not very successful in defending these suits; but, later, its engineers completed their studies of the situation and were able to prove that the farmers had suffered no damage, with the exception of such litigants whose farms lay directly alongside the aqueduct and within the cone influence of the pumps when operating.

A study of the tree growth within the area of the farms under litigation, as well as elsewhere in the United States, where rainfall and underground-water data were obtainable, made by the writer, established the fact that such growth was not affected by the pumping, and that as the water-table lay so far below the surface throughout the area under litigation that vegetation depended on rainfall for its growth.

There is an important question to be solved: How much of the rainfall, which is absorbed by the ground and finally reaches the ground-water, may be pumped safely without damage to the underground reservoir? This is especially important in cases such as Long Island, as the fresh water holds back the salt water, and if the latter is permitted to encroach on the former the volume of fresh water available will be reduced by that quantity irretrievably, and this is the real problem which confronts the engineer.

Mr. Conkling is to be congratulated on his very valuable and timely paper.

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DISCUSSIONS

BACK-WATER AND DROP-DOWN CURVES FOR UNIFORM CHANNELS

Discussion

By Boris A. Bakhmeteff, M. Am. Soc. C. E.

Boris A. Bakhmeteff,[12] M. Am. Soc. C. E. (by letter).[12a]—One cannot fail to be impressed by the painstaking efforts and the time which Professor Mononobe and his collaborators must have invested in the preparation of this paper. In the early stage of his studies on non-uniform flow[13] more than twenty-five years ago, studies which led to the methods subsequently developed in "Hydraulics of Open Channels",[14] the writer was confronted with the maze of the different formulas then extant and went through a process of comparison similar to that so ably presented in Professor Mononobe's work. Obviously, to make the general Bélanger formula (Equation (1)) usable for practical purposes, it must be integrated in relation to cross-sectional forms and resistance formulas as they actually occur in engineering practice. That requires, first, stepping outside the limits of the idealized broad rectangular and parabolic channel sections, as treated by the earlier pioneers, as well as doing away with the assumption of a constant Chézy factor value, C, as used by Bresse, Rühlman, and others.

Professor Mononobe correctly indicates that "monomial forms" should be used for this purpose, and recourse is taken to exponential expressions, which is in accordance with earlier suggestions of the writer, suggestions of which Professor Mononobe had no previous knowledge.

It appears to the writer, however, that the thought of giving monomial expressions separately to the area, wetted perimeter, and other elements, as suggested in Equations (13), is not so fortunate. In fact, in such general form, the assumption is somewhat misleading, and, moreover, it tends to eclipse the real significance of exponential expressions. The essence of the method lies in the fact, that any curve, $y = f(x)$, the value of which increases

Note.—The paper by Nagaho Mononobe, M. Am. Soc. C. E., was published in May, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1936, by J. C. Stevens, M. Am. Soc. C. E.; and February, 1937, by Messrs. Chesley J. Posey, and A. A. Kalinske.

[12] Prof., Columbia Univ., New York, N. Y.

[12a] Received by the Secretary February 5, 1937.

[13] "On Varied Flow of Liquids in Open Conduits" (in Russian), St. Petersburg, 1912.

[14] Engineering Society's Monograph, McGraw-Hill Co., 1932.

with x (Fig. 21), may be replaced within certain limits ($A - B$, Fig. 21), with sufficient approximation, by an exponential expression of the type, $y = a\,x^n$.

With reference to Equation (1), the areas and the other elements do not enter singly, but in the combination,

$$A\,C\,\sqrt{R} = K \qquad (39)$$

constituting thus a combined parameter, which connects the bottom slope with the discharge in uniform motion,

$$Q = K_0\,\sqrt{S_0} \qquad (40)$$

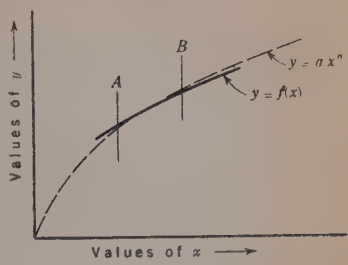

FIG. 21

and which the writer qualifies as the "carrying capacity."[15] This value, properly substituted into Equation (1) makes it,

$$S = S_0 - \frac{dD}{dx} = \frac{Q^2}{K^2} - \alpha\,\frac{S_0}{\sigma}\,\frac{K^2_0}{K^2}\,\frac{dD}{dx} \qquad (41)$$

in which σ is the critical slope.

For an open conduit, Equation (39) is a function increasing with the depth and, within certain limits, can be expressed monomially, in the sense of Fig. 21, leading to a simple relation,

$$\left(\frac{K_2}{K_1}\right)^2 = \left(\frac{D_2}{D_1}\right)^n \qquad (42)$$

with a single value of n = the hydraulic exponent.

In pursuing his course further, Professor Mononobe arrives at Equation (18), with integral functions of the type:

$$\Phi_1 = \int \frac{y^r}{y^r - 1}\,dy \qquad (43a)$$

and,

$$\Phi_2 = \int \frac{y^{(r-2S)-1}}{y^r - 1}\,dy \qquad (43b)$$

Practical application depends on knowing the numerical values of these functions and therefore, appropriate graphical charts are given in the paper. According to Professor Mononobe, numerical evaluation of the function values in certain cases was direct, which is assumed to mean quadratures, whereas in other cases recourse had to be taken to converging series. Now, in historical retrospect, Equation (18) is generally similar in form to the

[15] "Hydraulics of Open Channels", Engineering Society's Mongraph, McGraw-Hill Co., 1932, p. 13.

relations traditionally used by Dupuit, Bresse, and subsequent investigators. With Professor Mononobe's notation the basic relation,[16] Equation (4), which is equivalent to Equation (18), and in the form used by the writer would read:

$$\frac{S_o\ l}{D_o} = \left(\frac{D_b}{D_o} - \frac{D}{D_o}\right) - (1 - \beta)\left[B\left(\frac{D_b}{D_o}\right) - B\left(\frac{D}{D_o}\right)\right] \ldots\ldots (44)$$

in which $\beta = \frac{S_o}{\sigma}$ is taken to be a coefficient, whereas the varied flow function, B, is the numerical value of the integral,

$$B = -\int_o^y \frac{dy}{y^n - 1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (45)$$

an expression into which Professor Mononobe's function, Φ_1, is reduced by the simple transformation,

$$\int_o^y \frac{y^r}{y^r - 1}\,dy = \int_o^y \frac{y^r - 1 + 1}{y^r - 1}\,dy = y + \int_o^y \frac{dy}{y^r - 1} \ldots\ldots (46)$$

The substantial difference between Professor Mononobe's technique and that of the previous investigators (including the writer), is that, whereas heretofore the customary manner was to present the function values in tabular form, the paper gives them graphically in chart form. Obviously, tables and charts are equivalent, the preference for one or the other being a matter of individual taste. An illuminating analogy would be to replace the customary tables of logarithms by graphical charts.

In the light of the preceding, the writer does not quite understand in what way Professor Mononobe's approach avoids the mathematical complications supposedly inherent in earlier methods, and in what manner it simplifies computations as claimed.

With regard to details, confusion may be avoided by correcting possibly a typographical error, in using an identical symbol, C, in Equation (1) for the generalized Chézy factor, and for what is obviously meant to be a numerical constant in Equation (2).

The attempts of Professor Mononobe to complement his analysis with experimental work should be more than welcomed. In fact, although the varied-flow theory has been the subject of repeated treatment, scarcely any experimental data are available, the best probably being still those assembled in the classical opus by Darcy and Bazin.[17] The usual difficulty lies in the necessity of having very long flumes, and Professor Mononobe's manner of using short canals is ingenious. It is to be hoped that Professor Mononobe's experiments will once more draw attention to this much neglected field of research, and that, in particular, observations on a larger scale will become available.

[16] "Hydraulics of Open Channels", p. 88.

[17] "Recherches Hydrauliques", Paris, 1865.

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DISCUSSIONS

DYNAMIC DISTORTIONS IN STRUCTURES SUBJECTED TO SUDDEN EARTH SHOCK

Discussion

BY L. S. JACOBSEN, ESQ.

L. S. JACOBSEN,[13] ESQ. (by letter).[13a]—If a vibrating system with one degree of freedom experiences a transitory ground motion of constant period, but of variable amplitude, the dynamic distortions depend mainly on the ratio of the period of the ground to the one of the system. This fact has been demonstrated conclusively by Professor Williams. The paper shows by model experiments that for the case of near resonance large dynamic distortions of the system are produced even by a few oscillations of the ground.

The primary object of this discussion is to emphasize the absolute necessity of considering the dynamic properties of structures if a rational understanding of the problem of earthquake resistant design is to be evolved in the future. The secondary object of the discussion is to supply information concerning the dynamic distortions of a multiple-mass system as exemplified by a model of a 17-story building.

It is true that, at present, very few engineers can afford to make searching studies of the dynamic properties of the structures they design. Time limitations demand that so-called "rules of thumb" must be used for assuming the magnitudes of the dynamic loads. Because of this insufficient and imperfect knowledge of ground motions that occur during earthquakes, it is obvious that designers are not justified in making any specific assumptions in regard to the periods, durations, and intensities of the composite motions. Perhaps definite probable periods exist in particular localities, or perhaps the entire question of probable ground motions is so vague that no one can afford to risk making assumptions concerning the specific motions. Whatever their attitudes are toward the ground-motion question, they are forced to admit that forward and backward, as well as up and down, motions

NOTE.—The paper by Harry A. Williams, Assoc. M. Am. Soc. C. E., was published in May, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1936, by Messrs. Arthur C. Ruge, and H. M. Engle; and February, 1937, by Merit P. White, Jun. Am. Soc. C. E.

[13] Prof. of Mech. Eng., Stanford Univ., Stanford University, Calif.

[13a] Received by the Secretary January 15, 1937.

occur. This fact alone means that dynamic loadings varying with time must be sustained by the structures.

Professor Williams attempts to show the effects of a few specific ground motions in a horizontal direction on a single mass structure. It is of importance to note that the ground motion must be specific; otherwise, an elementary analysis is impossible. Moreover, if the ground motion were not specific and simple, it would not be feasible to extend and interpret the analysis by principles of superposition to more complicated types of ground motions. It is true, however, that when the results of the simple system and simple ground motion are applied to actual problems, the quantitative aspect of the analysis is lost and only the qualitative remains. It is precisely here that engineering judgment is necessary.

How is sound engineering judgment concerning seismic loadings to be formed? Naturally, one must admit that actual field evidence has given, and still gives, the first and indisputable clue as to what "stands up" and what "falls down", or what is seriously damaged. Under no circumstances can one afford to minimize the value of the expensive lessons given by "Nature's own laboratory." It is well known, however, that a group of engineers visiting the stricken region after an earthquake can, and does, find abundant evidence for supporting many different individual points of view. This is especially so if the damage is considerable so that the wreckage is largely in the form of heaps of bricks, concrete, tiles, and steel. Too many observers of earthquake damage lack something which is of extreme importance for the formation of sound engineering judgment, namely, a "pattern of characteristic behavior" of simple systems subjected to simple, although perhaps improbable, types of ground motion. Before it is possible to estimate how a complicated phenomenon has occurred one must have not only a picture, but also a quantitative picture of the simple phenomenon. Professor Williams has attempted to give such a "pattern of behavior", for which he deserves credit.

Since the customary practice of engineers is to design structures to resist fictitious, horizontal, static loadings expressed in percentages of the static, vertical loads, or in terms of horizontal, constant accelerations of the ground, it has occurred to the writer that a slight re-arrangement of the author's results may show more clearly the relation of the truly dynamic loadings to the fictitious, static loadings. It must be remembered that, in Professor Williams' experiments, the dynamic loadings have been inferred from the actually observed dynamic distortions; they are real loads, and, therefore, can not be explained away by the all too frequently heard remarks that such dynamic loads will not have time to become real loads and have the same effect as static loads.

Results obtained from Fig. 3 of the paper have been re-arranged as shown in Table 1, with the following supporting data:

(1) The properties of the model are: Weight, 25.5 lb; spring factors, 17.3, 9.2, and 7.1 lb per in.; friction factors, 0.05, 0.10, and 0.15 lb per in. per sec; and free vibration periods, 0.390, 0.534, and 0.611 sec.

(2) For ground motion, or shaking-table motion: The period is 0.505 sec; the duration of motion is about 5.5 sec; the maximum impact acceleration is about 14% of gravity; the maximum harmonic acceleration is about 8.5% of gravity; the maximum displacement is about 0.25 in.; and, the constant friction reduces the amplitude of the table by approximately 0.022 in, per cycle.

TABLE 1.—RE-ARRANGEMENT OF RESULTS OBTAINED FROM FIG. 3 OF THE PAPER

	BELOW RESONANCE			NEAR RESONANCE			ABOVE RESONANCE		
Ratio of model period to ground period	0.77			1.06			1.21		
Ratio of consecutive, free, damped amplitudes of model corresponding to the three values of damping used by Professor Williams............	1.16	1.34	1.55	1.22	1.49	1.83	1.26	1.58	1.99
Ratio of maximum dynamic force on model to weight of model..........	0.38	0.30	0.27	0.58	0.41	0.29	0.24	0.20	0.17
Number of cycles of ground motion when the maximum dynamic force occurs..........................	1.8	1.7	1.6	5.1	4.3	3.9	2.8	2.6	2.4
Ratio of maximum dynamic force to a constant fictitious force of same value as the maximum harmonic acceleration of the ground, 0.085 g times mass of the model..........	4.5	3.5	3.2	6.8	4.8	3.4	2.8	2.4	2.0

For the three cases investigated Table 1 shows that the ratios of the maximum dynamic forces, acting on the model as a result of the motion of the ground, are greatly in excess of the fictitious static forces which are equal to the mass of the model times the maximum, harmonic acceleration of 8.5% of gravity. In other words, the inherent vibrational properties of the model for the three cases are such that, with the minimum friction used, a

TABLE 2.—SHOWING MASS AND ELASTIC PROPERTIES OF THE SEVENTEEN-STORY MODEL (FIG. 11), TOGETHER WITH RATIOS OF EXPERIMENTAL MAXIMUM DYNAMIC SHEARS AS COMPUTED FROM THE RECORDS OF FIG. 3.

Floor No.	Weight of floor, in pounds	Rigidity of floor, in pounds per inch	SHEAR RATIOS: Impact / 0.32 g	SHEAR RATIOS: Maximum dynamic / 0.20 g	Floor No.	Weight of floor, in pounds	Rigidity of floor, in pounds per inch	SHEAR RATIOS: Impact / 0.32 g	SHEAR RATIOS: Maximum dynamic / 0.20 g	Floor No.	Weight of floor, in pounds	Rigidity of floor, in pounds per inch	SHEAR RATIOS: Impact / 0.32 g	SHEAR RATIOS: Maximum dynamic / 0.20 g
(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
17	3.24	13.6	0.19	4.95	11	2.65	19.7	0.41	1.70	6	2.65	24.9	0.28	0.55
16	2.55	14.6	0.31	4.15	10	2.65	20.8	0.25	1.40	5	2.65	25.9	0.28	0.80
15	2.48	19.2	0.29	2.90	9	2.65	21.8	0.31	0.65	4	2.65	26.8	0.38	0.90
14	2.48	20.5	0.22	2.25	8	2.65	22.8	0.60	0.90	3	2.65	27.8	0.31	0.90
13	2.65	17.6	0.38	3.00	7	2.65	23.8	0.34	0.55	2	2.65	28.8	0.39	1.15
12	2.65	18.6	0.62	2.80	6	2.65	24.9	0.28	0.55	1	2.65	∞		
11	2.65				5	2.65								

force of 4.5 times the fictitious, static force results for the first case, or the "below resonance" case, whereas a force of 6.8 times the fictitious, static force is experienced by the model in the second or "near resonance" case; and, finally, a force of 2.8 times the fictitious, static force is in action for the third or "above resonance" case. If one notes that the number of cycles of ground motion necessary to establish the distortions corresponding to the foregoing dynamic forces are 1.8, 5.1, and 2.8, respectively, one is forced to conclude that the first case (or the "below resonance" case) assumes a sinister aspect since in an actual earthquake the probability of encountering a constant periodic motion lasting for 1.8 cycles is quite high. For the three cases, with the maximum friction used, the force ratios are 3.2, 3.4, and 2.0, whereas the numbers of cycles necessary for producing the distortions corresponding to the forces are 1.6, 3.9, and 2.4. Furthermore, in this case, the "below resonance" condition seems to be rather dangerous.

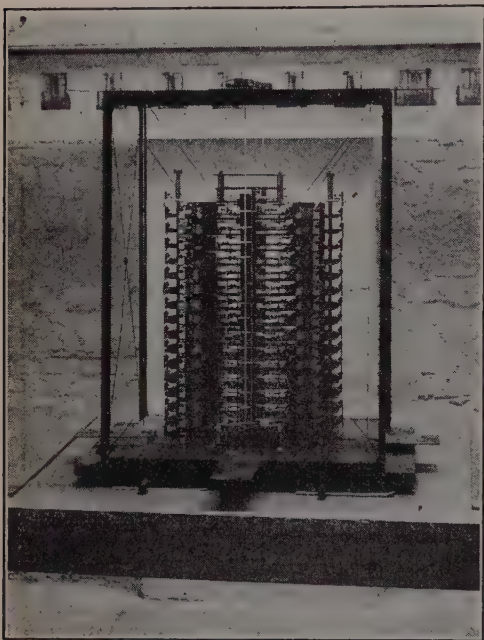

Fig. 11.—Model of 17-Story Building.

The first three columns of Table 2 give information concerning the mass and elastic shear properties of the 17-story model shown in Fig. 11. The model, built in 1931, was tested in 1932–33. Since the construction of this model allows only distortions due to shear and neglects those due to

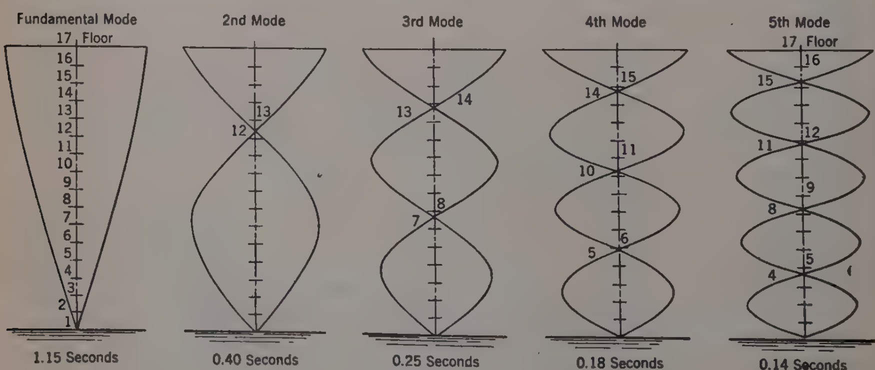

Fig. 12.

flexure of the building as a whole, the experiments have never been published before. It can be shown, however, that in the case of a rather stubby building, flexure of the building as a whole may be neglected, so that if the following model experiments are restricted to stubby buildings no serious error is committed.

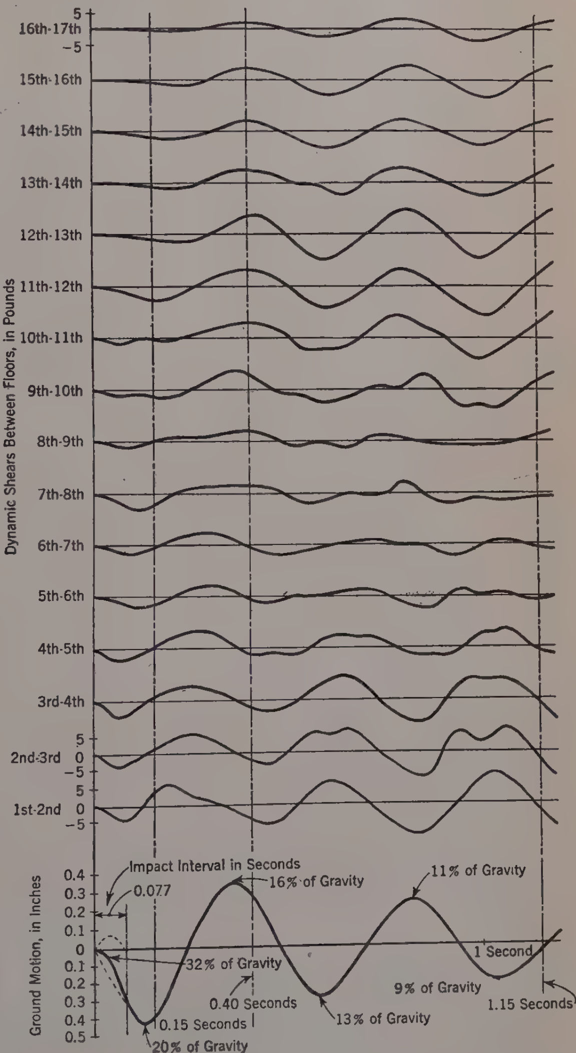

FIG. 13.

Fig. 12 shows the calculated displacement diagrams of the first five modes of free vibration of the model. The free vibration periods of these modes are almost in the simple ratios, 3, 5, 7, and 9, corresponding to a model with constant mass and shear rigidity distribution. The maximum shear distortions occur at the nodal points. Thus, for the second mode, the maximum shears occur between the 1st and 2d floor and between the 12th and 13th floor, whereas for the fifth mode, the maximum shears occur between the 1st and 2d, the 4th and 5th, the 7th and 8th, the 11th and 12th, and the 14th and 15th floors.

Fig. 13 shows sixteen time records of the dynamic forces acting between the seventeen floors of the model when the ground motion is given by the time displacement records below it. The ground motion is generated, as in Professor Williams' experiments, by the impact of a pendulum against a bumper spring on the shaking-table. During the impact interval (that is, the time during which the pendulum is in contact with the bumper spring), the ground motion may be thought of as being composed of two component motions, the

primary motion of 0.15-sec period, lasting for about one-half cycle and the secondary motion of 0.46-sec period, lasting for about one-sixth of a cycle, or until the instant when contact with the pendulum ceases. At this instant the motion of the shaking-table becomes a free, damped vibration with a period of 0.47 sec, and this motion lasts for several cycles until friction in the shaking-table makes it cease. Nearly all the sixteen records in Fig. 13 give evidence of the existence of the two ground periods, 0.15, and 0.47 sec, in the dynamic force fluctuations.

The reasons for the pronounced effects are that the natural period of 0.14 sec of the model's fifth mode of vibration is in "near resonance" with the period of 0.15 sec of the primary impact motion of the shaking-table. In spite of the fact that this ground motion exists for only one-half cycle, its effect is observable throughout the records. Moreover, the secondary ground motion of 0.46-sec period, or the free vibration ground motion of 0.47-sec period, is also in "near resonance" with the model's second mode period of 0.40 sec. The effect of this "near resonance" is a building up of the dynamic forces with time, especially at the nodal points; that is, between the 1st and 2d, and between the 12th and 13th floors. Almost no dynamic forces of this period occur at the anti-nodal point between the 7th and 9th floors.

Columns (4) and (5), Table 2, give the measured, dynamic, maximum forces between adjacent floors divided by the fictitious, static forces corresponding to an assumed constant acceleration of the model. Column (4) relates to the maximum loads experienced by the floors during the first 0.15 sec of the ground motion. They are compared to the fictitious, static loads corresponding to a constant ground acceleration, equal to the maximum impact acceleration of 32% of gravity. These ratios are all less than unity, signifying that the full effects of such a rapid ground motion are not established in the model. It should be noted, however, that the effects are maxima between the 3d and 4th, between the 7th and 8th, between the 11th and 12th, and between the 15th and 16th floors. These locations agree very well with those of the nodal points of the fifth mode of free vibration of the model.

Column (5), Table 2, gives the ratios of the maximum, "built-up", dynamic forces experienced by the floors during the first 1.15 sec of the ground motion to the fictitious, static forces corresponding to a constant acceleration of the ground of 20% of gravity. The latter gravity acceleration is equal to the maximum harmonic acceleration of the ground occurring at the peak of the first swing. Most of the ratios are greater than unity for the reason that a "building up" has occurred even if only about two complete cycles of the ground motion have taken place. In this case, also, the maxima are located at the theoretically correct places, namely, between the 1st and 2d and between the 12th and 13th floors. The rapidly increasing ratios of the forces at the top of the model are due to the fact that for the customary calculations of fictitious, static forces only the mass of the model above a floor is effective in producing the shears.

If it might be assumed that a building always were to vibrate solely in its fundamental modes, the customary fictitious static force calculations would

have a reasonably theoretical backing; but since such an assumption is not at all warranted, either theoretically or by experience, the customary "rules of thumb" relating to the fictitious, static force assumptions are without a rational basis.

It appears from this test, and from many tests yet unpublished, that there is no reason for assuming that the probable dynamic shear in a building exposed to a ground disturbance decreases from the top toward the bottom of a building. A much more reasonable assumption is that the probable dynamic shears between adjacent floors are constant throughout the building.

This test and many others on multiple-mass systems corroborate Professor Williams' conclusions in regard to the single mass system. Large dynamic forces can be produced by one or two cycles of ground motion in "near resonance" with one of the natural periods of the multiple-mass system.

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DISCUSSIONS

ANALYSIS OF VIERENDEEL TRUSSES

Discussion

BY MESSRS. KIMBALL R. GARLAND, AND J. D. GEDO

KIMBALL R. GARLAND,[43] ESQ. (by letter).[43a]—A series of equations is presented in this paper that should be of great aid in analyzing Vierendeel trusses, and the author deserves much credit for having presented the method and results in such complete form that they can be followed through and checked. Exact methods for solving this type of truss have been lacking in American literature, and a need for such methods (which this paper will help to supply) is growing. It is unfortunate that, in spite of the careful analysis which has been made of this type of truss, the solution is still extremely laborious. In the case of the truss with horizontal bottom chords and inclined top chords, even with the equations presented, and with the simple example which was solved in the text, the labor of obtaining a result is considerable. In the case of practical design, where many other complications must be considered, and several trial designs made, many American offices, in the writer's opinion, would consider the difficulty of solution as a weighty argument against its use.

Some years ago, the writer made some comparative designs for a building foundation which was intended to be rigid enough to resist the effects of unequal soil-bearing pressure, utilizing the entire basement story as a framework. The Vierendeel truss seemed to be the proper type of design for this purpose, so that the basement might be free from the obstruction of diagonal members. On this occasion, a set of equations, such as the author has presented, would have been of great value. Not having any literature immediately available on the subject, the writer made a number of solutions based on the method of moment distribution, and found that results could be obtained of sufficient accuracy for the purpose, although the work involved was rather laborious. One design which was studied had very large haunches at the top and bottom joints, and the horizontal members were also tapered between the haunches, for economy, being thinner near the middle of the panel

NOTE.—The paper by Dana Young, Assoc. M. Am. Soc. C. E., was published in August, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1936, by Messrs. L. J. Mensch, A. A. Eremin, Leon Blog, A. W. Fischer, and L. C. Maugh; January, 1937, by John E. Goldberg, Jun. Am. Soc. C. E.; and February, 1937, by Messrs. Louis Baes, E. C. Ingalls and Ralph B. Peck, Harold E. Wessman, and A. Floris.

[43] Newton, Mass.

[43a] Received by the Secretary January 20, 1937.

where the bending was assumed to be least. This introduced a complication, in that the axis of all the horizontal members was bent in a vertical plane, so that the bending moment in these members was dependent in part on the axial stress in the chord, as well as on the shear which the panel carried. The process followed was to plot what was assumed to be the axis of each member, and determine its stiffness coefficient as if its axis were straight. Then, for the first trial analysis, it was assumed that this axis was straight, extending between the points where vertical and horizontal members intersected, and a solution was made by the method of moment distribution. The resulting axial stresses in the chords produced additional moments due to the eccentricity of the thrust, and a second moment distribution was made to correct for this. This cycle, in turn, changed the chord stresses, requiring a third correction, which seemed to have carried the process far enough. The proper method in this case would have been to substitute for the bent axis of each chord a straight axis, located so that a thrust or a tension acting along it would produce no angular displacement in either end of the member; or, in other words, through the center of gravity of the values of $y \frac{ds}{E I}$ for that member. After this is done, one analysis by moment distribution should suffice to determine the true values. Once this shifting of the axis of the member is accomplished, the author's method might be applied, although this would require further investigation. In any case, it seems as if some kind of correction should be applied where trusses have large gussets.

One uncertainty which makes it unwise to strive for too great refinement in the stress analysis of Vierendeel trusses of concrete, is the lack of knowledge regarding the tension chord. Should the reinforcement alone be computed, where both top and bottom rods are in tension, or should some weight be given to the concrete, which must certainly carry some stress, and, therefore, must add to the stiffness of the member?

As a matter of curiosity, the writer made a solution of the truss which the author solved in Examples 3 and 5 by the method of moment distribution. He made the usual assumption that every joint is held from rotating, although it may be displaced horizontally or vertically. He assumed, furthermore, that Vertical No. 3 took no horizontal shear, but that the members to the left took up all the thrust to the left, and those to the right took all the thrust to the right, due to the distortion of the inclined chords. It was then possible to write the expressions for the moments and shears in the two left panels in two simultaneous equations, and solve for the values. The same could be done for the two panels at the right. The center panel, being rectangular, was solved by inspection. The method of moment distribution was then applied. The steps would be as follows: (1) Solve for unbalanced vertical shear (two sets of short simultaneous equations); (2) balance joints; (3) carry over moments; (4) distribute the unbalanced horizontal shear among web members; and, (5) find the unbalanced vertical shear in each panel, and solve again by simultaneous equations.

The writer found that the moments converged consistently by this method, and that in five repetitions the results checked fairly closely (about 1%) with the answers given in the problem. This method, of course, was laborious. After working through the solution given in the paper for the same problem by the general method, the writer thinks that probably, for only one condition of loading, and where an accuracy of 1% is sufficient, the moment-distribution method would be as rapid as the solution by the general equations. For complete analysis of all possible conditions of loading, the general solution would save time in the end, and would give the exact results.

J. D. GEDO,[44] M. AM. SOC. C. E. (by letter).[44a]—The novelty in Professor Young's solution is that equations similar to those of the theorem of three moments may be written for the column shears. Thus, before any other unknowns are solved, the horizontal stresses are obtained from Clapeyronic equations. This is a valuable contribution.

It is surprising, however, that he under-estimates the value of the basic formulas, Equations (32), (37), and (42), in that he states: "A set of these formulas could be written for each panel and the series solved simultaneously to find the unknowns. Such a procedure, although possible, would be too laborious for practical use." As a matter of fact, Professor Young does write the three equations for each panel. Were these 3 n-equations such that they contain the 3 n-unknowns, not only collectively but also individually, the solution would be impossible in all but the simplest cases. The success of his analysis is not due entirely to the manipulation of these valuable equations; the secret is that when integrating the product of the virtual stresses and real deformations the integrations do not cover the entire structure, they extend only to the panel. Professor Young does not make any effort to explain why this is permissible; nor does he emphasize the importance of this procedure. It is true, however, that he refers to the work of J. A. Van den Broek,[6] M. Am. Soc. C. E., which contains such explanations.

Professor Young is misled when he assumes that, by solving the series simultaneously, the procedure would be too laborious for practical use. The writer has computed many Vierendeel trusses with similar elastic equations, and he has always found that it is possible to compute the influence lines of 10-panel Vierendeel trusses with horizontal or inclined upper chords in two working days, provided the truss is symmetrical about a vertical axis. With Professor Young's contribution this time is shortened by two hours.

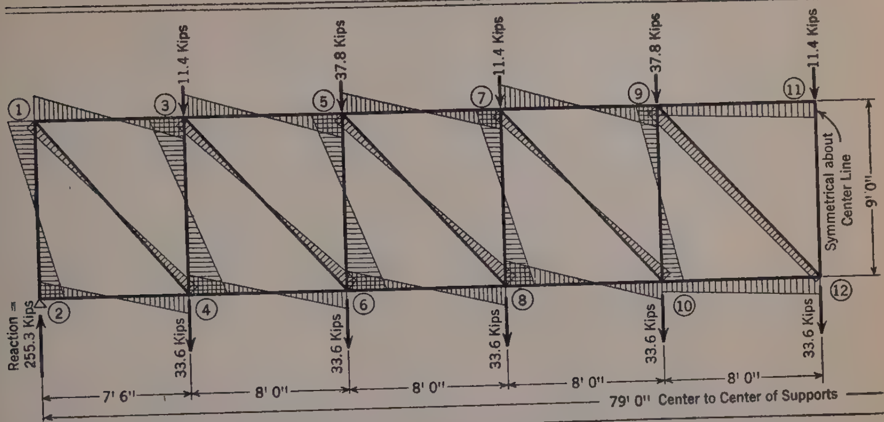
Statements with which the writer does not agree are: That Vierendeel trusses may be built of reinforced concrete and that the effect of axial deformation is so small that it may be neglected in most cases. The example in Table 3 was computed to "throw light" on these statements and also to compare the Vierendeel truss with the closely related pinned truss and riveted truss. In the diagram of Table 3, the moments are shown on the sagging side of the system axis. The frame was designed as a Vierendeel truss and

[44] Senior Designer, Park Dept., City of New York, New York, N. Y.

[44a] Received by the Secretary February 8, 1937.

[6] "Elastic Energy Theory", by J. A. Van den Broek, J. Wiley & Sons, New York, 1931.

TABLE 3.—COMPARISON OF STRESSES IN VIERENDEEL, RIVETED, AND PIN-CONNECTED TRUSSES



General			Bending Moment, in Foot-Kips			Direct Force, in Kips				Deflection, in Inches			
Member	Moment of inertia, in $inches^4$	Area, in $inches^2$	Vierendeel truss (direct forces neglected)	Vierendeel truss (direct forces considered)	Riveted truss	Vierendeel truss (direct forces neglected)	Vierendeel truss (direct forces considered)	Riveted truss	Pin-connected truss	Location	Vierendeel truss (direct forces considered)	Riveted truss	Pin-connected truss
1-2	2 798	64.36	476.6	464.6	155.7	127.7	125.9	207.7	255.3	..	.....	.....	.....
1-3	2 798	64.36	476.6	464.6	164.1	105.9	104.3	173.1	212.8	..	.....	.....	.....
1-4	797	21.76	.....	.....	8.4	......	......	—212.7	—332.3	..	.....	.....	.....
2-1	.....	.....	476.5	474.4	167.6	......	......	......	......	2	0.00	0.00	0.00
2-4	2 798	64.36	476.5	474.4	167.6	—105.9	—104.3	—35.9	......	..	.....	.....	.....
3-1	.....	.....	481.1	479.4	174.6	......	......	......	......	..	.....	.....	.....
3-4	5 454	108.78	907.6	890.4	302.4	—11.2	—8.4	140.2	221.7	..	.....	.....	.....
3-5	2 798	64.36	426.5	411.0	136.0	307.6	303.0	362.9	399.7	..	.....	.....	.....
3-6	642	17.94	.....	.....	8.2	......	......	—182.4	—281.4	..	.....	.....	.....
4-1	.....	.....	.....	.....	7.1	......	......	......	......	4	0.37	0.23	0.27
4-2	.....	.....	480.6	496.4	189.2	......	......	......	......	..	.....	.....	.....
4-3	.....	.....	907.4	897.8	308.6	......	......	......	......	..	.....	.....	.....
4-6	2 798	64.36	426.8	401.4	126.5	—307.6	—303.0	—241.0	—212.8	..	.....	.....	.....
5-3	.....	.....	414.5	437.4	170.1	......	......	......	......	..	.....	.....	.....
5-6	3 912	84.37	696.0	686.0	237.0	2.2	0.7	112.2	176.7	..	.....	.....	.....
5-7	2 798	64.36	281.5	248.6	76.7	462.2	455.4	494.4	523.2	..	.....	.....	.....
5-8	432	12.34	.....	.....	9.9	......	......	—117.0	—185.8	..	.....	.....	.....
6-3	.....	.....	.....	.....	3.3	......	......	......	......	6	0.73	0.46	0.55
6-4	.....	.....	414.6	432.5	164.5	......	......	......	......	..	.....	.....	.....
6-5	.....	.....	695.9	685.4	239.0	......	......	......	......	..	.....	.....	.....
6-8	2 798	64.36	281.3	252.9	77.7	—462.2	—455.4	—415.7	—399.7	..	.....	.....	.....
7-5	.....	.....	274.3	302.6	131.0	......	......	......	......	..	.....	.....	.....
7-8	2 402	56.73	443.3	437.4	160.6	—11.1	—9.7	60.0	105.3	..	.....	.....	.....
7-9	2 798	64.36	168.9	134.8	38.0	560.7	552.6	579.1	606.6	..	.....	.....	.....
7-10	290	8.81	.....	.....	8.3	......	......	—72.8	—125.6	..	.....	.....	.....
8-5	.....	.....	.....	.....	4.3	......	......	......	......	8	1.04	0.65	0.78
8-6	.....	.....	274.1	307.0	132.2	......	......	......	......	..	.....	.....	.....
8-7	.....	.....	443.2	437.0	160.3	......	......	......	......	..	.....	.....	.....
8-10	2 798	64.36	169.1	130.0	32.4	—560.7	—552.6	—530.0	—523.2	..	.....	.....	.....
9-7	.....	.....	206.7	247.4	127.7	......	......	......	......	..	.....	.....	.....
9-10	1 166	30.26	173.4	170.9	61.2	2.1	—0.1	37.5	60.3	..	.....	.....	.....
9-11	2 798	64.36	33.3	76.5	58.5	599.2	590.7	609.4	626.6	..	.....	.....	.....
9-12	290	8.81	.....	.....	8.0	......	......	—24.4	—30.1	..	.....	.....	.....
10-7	.....	.....	.....	.....	2.3	......	......	......	......	10	1.27	0.78	0.97
10-8	.....	.....	206.5	239.0	122.4	......	......	......	......	..	.....	.....	.....
10-9	.....	.....	173.4	172.0	62.7	......	......	......	......	..	.....	.....	.....
10-12	2 798	64.36	33.1	67.0	57.4	—599.2	—590.7	—592.8	—606.6	..	.....	.....	.....
11-9	.....	.....	122.9	155.5	78.6	......	......	......	......	..	.....	.....	.....
11-12	290	8.81	.....	.....	.....	—11.0	—8.4	6.4	11.4	..	.....	.....	.....
12-9	.....	.....	.....	.....	1.5	......	......	......	......	12	1.35	0.83	1.01
12-10	.....	.....	123.5	168.0	74.4	......	......	......	......				
12-11	.....	.....	.....	.....	.....	......	......	......	......				

$E = 30\ 000\ 000$ lb per sq in.

the sections were retained in the case of the other trusses. Two analyses were made for the Vierendeel truss. In the second analysis the axial deformations were considered not only in the chords but also in the posts. Hence, the dissymmetry between top-chord and bottom-chord stresses. Comparing the two analyses it is evident that the effect of the direct forces on the moment distribution is far too great to be negligible. It is also evident that the truss in Table 3 can not be built of reinforced concrete. The direct tension in Member 10–12 is nearly 600 kips. This force alone would require about forty 1-in. square bars. The shear in Member 3–4 is about 200 kips. Allowing 100 lb per sq in. for shear, the area of the section would be 2 000 sq in. Thus, it is easy to see that reinforced concrete is not the proper material for a Vierendeel truss except, possibly, for very small ones.

Although the subject of this paper concerns only the stress analysis, the results can not be properly interpreted without comparing the Vierendeel truss with frames that resemble it. In Table 3 it is noticeable that the moments of the Vierendeel truss are much higher than the so-called "secondary stresses" of the riveted truss. The writer has not analyzed the riveted truss by the slope deflection method, the stresses were obtained by using the same reasonings as in the case of the Vierendeel truss. The direct forces are smallest in the Vierendeel truss, higher in the riveted truss, and highest in the pinned truss. Adding the diagonal material to the chords and posts, the sections thus increased may suffice for the higher stresses in the Vierendeel truss.

Furthermore, the stresses in the Vierendeel truss can be reduced by spacing the posts closer together near the supports and farther apart near the center. In other words, the ideal spacing for the posts is similar to that of the stirrups in a uniformly loaded reinforced concrete beam. In interpreting the relatively high deflections of the Vierendeel truss, it should be borne in mind that the dimensions of both the riveted and the pinned truss are greater than necessary, in the example. No doubt, many more comparative studies are needed to arrive at a definite conclusion. Nevertheless, it is certain that the Vierendeel truss has a great future. Professor Young's contribution, therefore, is doubly meritorious, although some of his minor statements are not quite exact.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

SIMPLIFIED METHOD OF DETERMINING TRUE BEARINGS OF A LINE

Discussion

BY MESSRS. ROBERT H. MERRILL, CROSBY J. WILKIN, AND C. S. JARVIS

ROBERT H. MERRILL,[24] M. AM. SOC. C. E. (by letter).[24a]—By way of revising "tables in their most convenient form, as an Appendix", with due allowance for second differences, Fig. 3 shows (in the four margins) the four rates per minute of arc that are involved. In plotting the two columns under each degree of latitude from Table 1, many points fail to fall precisely on line. To secure uniformly spreading curves, these columns (to be eliminated by a graph) should be recalculated accurately to an additional decimal place.

For comparison the rates used in the typical example given in the paper are tabulated in Column (2) of the following example; the rates at the terminal values, $h = 25° \ 25' \ 30''$ and $\phi = 38° \ 53' \ 40''$ in Column (3) are also not necessary. Only at the mid-points of the interpolation (namely, at $h = 25° \ 13'$ and $\phi = 38° \ 27'$ are the true values found which are listed in Column (4).

Rate of: (1)	In:	From: (2)	To: (3)	Mean: (4)		Minutes added to h=25° and ϕ=38° (5)		A 1.39989 (6)	B 0.36378 (7)
A	h	19.4	20.2	19.8	×	25.5	=	505	...
A	ϕ	32.6	34.3	33.5	×	53.7	=	1 789	...
B	h	27.9	29.1	28.5	×	25.5	=	...	726
B	ϕ	22.1	23.2	22.6	×	53.7	=	...	1 212
								1.42292	0.38316

$d = -$ 1° 02′ 16″ nat sin = × 0.01811 = 0.02580

$Z =$ 65° 51′ 38″ nat cos = 0.40896

Instead of $Z =$ 65° 53′ 18″ (of the paper) = 0.40852

and since $Z =$ 65° 52′ 30″ by more precise calculation, the range in using Table 1 is + or − more than 30″.

NOTE.—The paper by Philip L. Inch, Assoc. M. Am. Soc. C. E., was published in September, 1936, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: November, 1936, by Messrs. Earl F. Church, Paul E. Wylie, James B. Goodwin, C. H. Swick, Philip Kissam, and George D. Whitmore; December, 1936, by Messrs. O. H. Chilton, Chalmers C. Schrontz, Frank M. Johnson, Walter H. Starkweather, and C. I. Day; January, 1937, by Messrs. F. L. McRee, F. J. Duarte, and Leonard C. Jordan; and February, 1937, by Messrs. J. C. Pinney, R. L. Vaughn and John C. Penn.

[24] Cons. Engr. (Spooner & Merrill), Grand Rapids, Mich.

[24a] Received by the Secretary December 28, 1936.

The late James B. Davis, M. Am. Soc. C. E., explained forty years ago "the mystery as to why the solar attachment has not come into more general use" by emphasizing the delicacy of adjustments, the extra weight, and the cost,

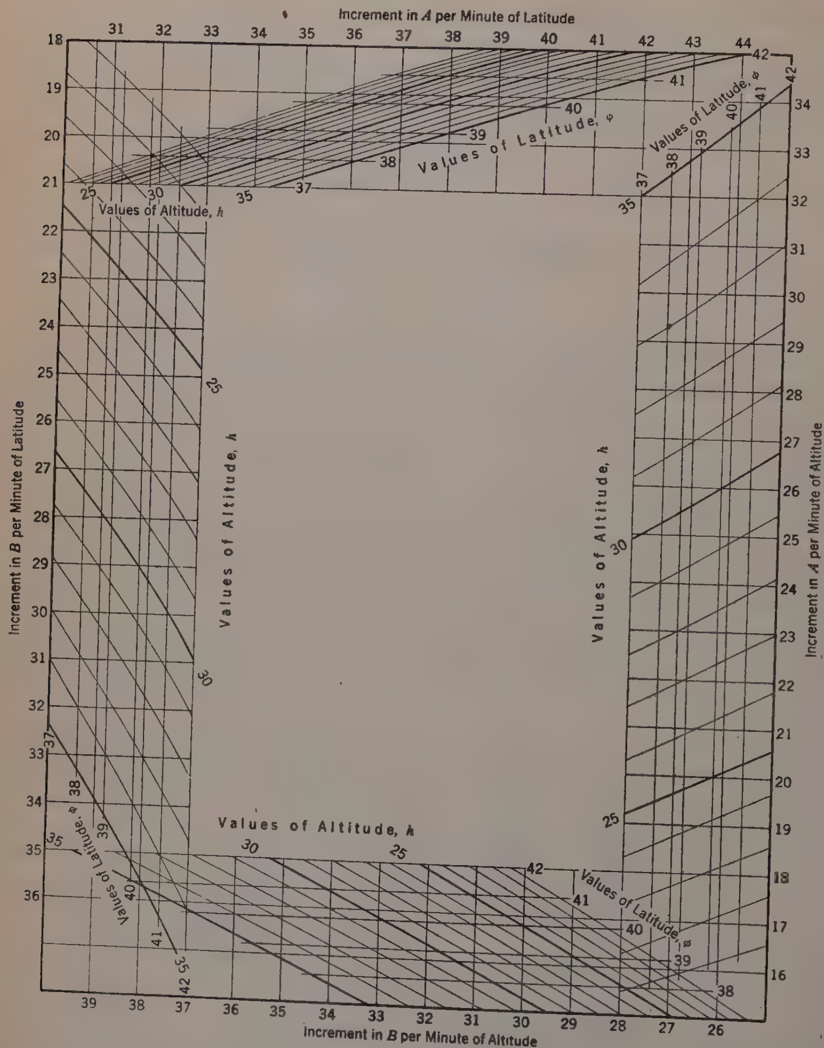

FIG. 3.

which is quite out of proportion to its infrequent use. About twenty-three years ago a "mechanical navigator"[25], costing $2 400 was placed on the market. It solved, mechanically, the general spherical triangles, but was heavy and bulky, with seven axes measuring altitude, latitude, hour angle, azimuth,

[25] *Engineering News*, Vol. 71, p. 180.

declination, and 90° on 6-in. disks with verniers that were precise to $\frac{1}{1200}$ in. Obviously, great mechanical skill was attained in adjusting all parts so perfectly that the last setting gave results accurate to 1′ or 2′ of arc. With a relatively cheap 10-in. straight slide-rule, or a 7-in. circular slide-rule, one may attain similar accuracy in solving the same problems.

For greater precision it is easy to recompute by the rather clumsy but adequate formula, using both naturals and logarithms of standard field tables thus, to solve the formula, $\cos Z = \dfrac{\sin d - \sin h \sin \phi}{\cos h \cos \phi}$:

Quantity	Angle	nat sin	log sin	log cos
d (−)	1° 02′ 16″	−0.01811	....	....
h	25° 23′ 37″	....	9.632290	9.955872
ϕ	38° 53′ 40″	....	9.797882	9.891149
$\sin h \sin \phi$	....	0.26926	9.430172	....
$\cos h \cos \phi$	....	....	....	9.847021
Numerator	....	−0.28737	....	9.458441
Z	65° 52′ 32″	....	....	9.611420

The writer favors the sun-disk bisections for observations on the sun as the easiest and the one developing all the accuracy of which an ordinary 1′ transit is capable. In the absence of colored-glass shades and a prismatic eye-piece, it is often handy to use a carpenter's 24-in. folding rule to hold the focusing card. With a spring clip near the 9-in. mark, clamp the 6-in. to 12-in. leg of the rule to the left front standard of the transit. Flex the rule at the 12-in. and the 18-in. joints so that the 21-in. mark falls on the line with the telescope about 3 in. below the eye-piece, and clip on the card at that point.

Although it is not necessary to use the uniform time intervals it is easy to check as the observations progress, according to the following schedule:

Watch Time: Hours	Minutes	Seconds	z	dz	h	dh
4	03	00	276° 03′		31° 10′	
				0° 04′		0° 13′
4	04	00	276° 07′		30° 57′	
				0° 05′		0° 14′
4	05	00	276° 12′		30° 43′	
				0° 04′		0° 13′
4	06	00	276° 16′		30° 30′	
				0° 04′		0° 15′
4	07	00	276° 20′		30° 15′	

The uniformity of the increments, per unit of time, shows at a glance that the middle pointing of the five was a fair mean value in spite of the erratic intervals. It is customary, however, to require the series of pointings to be continued until five consecutive ones show equal increments to the nearest 1′. Note also that, if well chosen, the more rapid motion in *dh* lessens the probable error in computed azimuth.

CROSBY J. WILKIN,[26] ASSOC. M. AM. SOC. C. E. (by letter).[26a]—Textbooks, slide-rules, tables, etc., are of little avail in obtaining the correct true bearings of a line if the data used in the formula are not right. The writer has yet to see a typical problem fully solved by the use of commonplace data. In his work, he would prefer to use true azimuths in all property surveys but he is somewhat hesitant to do so without considerably more experience to gain complete confidence in his computations; and he can imagine that surveyors with less education would be more non-plussed. This is probably the main reason why most surveys are based on a magnetic north, which is unreliable for re-running surveys, large or small, particularly in the absence of sufficient monuments.

The following solution was made on a survey in the summer of 1936. An engineer's transit was used in the field work and the writer obtained the data without assistance. The sun's image was focused on a white card wired to the telescope and the cross-hairs were made tangent to the image.

For the calculations, the Ephemeris of the Sun and Polaris, obtained from the U. S. Government Printing Office, was used, together with a calculating machine. The method is a composite of various other methods none of which was complete in itself for the tables at hand.

Field Work: Date, July 22, 1936; long., W 74°-08.4′; lat., 41°-38.1′; watch slow 9 sec.; temperature, 75° F:

Observation	Time A.M.	Horizontal angle	Vertical angle
Direct	8:54:35	115° -42′	44° -46′
Direct	8:56:04	116° -00′	45° -02′
Direct	8:56:55	116° -12′	45° -12′
Direct	8:57:42	116° -22′	45° -20′
Reversed	9:01:10	296° -19′	46° -31′
Reversed	9:02:35	296° -38′	46° -46′
Reversed	9:03:28	296° -49′	46° -55′
Reversed	9:04:28	297° -02′	47° -05′
Average	8:59:37.1	116° -23′.0	45° -57′.1

Computations:

Correction	+9. watch slow	− 0′.8 parallax and refraction
Corrected,	8:59:46.1 A.M.	45° -56′.3
Standard time of observation	8:59:46.1 A.M.	
Difference in time from standard time meridian, 75° −(74° -08′.4) = 0° -51′.6; or 0.86 × 4 = 3.44′ = 4 min for a degree of longitude	+3:26.4	
Local mean time of observation	9:03:12.5	
Equation of time	−6:17.3	

[26] Land Surv., Gardiner, N. Y.

[26a] Received by the Secretary December 30, 1936.

Apparent time of observation	8:56:55.2 = 3:03:04.8 before noon
Longitude reduced to time (74° -08′.4) ÷ 15 = or difference in time from Greenwich noon and place of observation.	4:56:33.6
Time of observation before noon	−3:03:04.8
Time elapsed since Greenwich noon	1:53:28.8 = 1.892 hr
Apparent declination at Greenwich noon	N + 20° −16.1′
Difference for 1 hr = 0.50′	
Difference for difference in time, 0.50 × 1.892 =	− 0.95
Declination for place of observation	20° -15.15′

$$\cos Z = \frac{\sin d - \sin h \sin l}{\cos h \cos l} = \frac{-0.13127}{0.51976} = 0.25276, \text{ or } Z = -75° \text{ -}22'$$

from south point, in which:

$d = 20°$ -15.15′ $h = 45°$ -56.3′ $l = 41°$ -38.1′

Horizontal angle based on magnetic needle	116° -23′
True azimuth of sun 180° − 75° 22′	104° -38′
Declination of needle	11° -45′ W.

The writer has no criticism to offer of the method proposed by Mr. Inch, for he gladly adopts simplification of any process in solving a problem which is as accurate as the usual method. However, the addition of a set of tables to the Ephemeris of the Sun and Polaris might only confuse the computer further. It seems to the writer that engineers had better continue using the customary method, eliminating any steps in the solution compatible with the precision required in the answer.

The difficulty with the usual solution is that, apparently, no one has endeavored to publish a method using the ephemeris only. The formula introduced in the foregoing example is easy to use with any calculating device, or even by long hand.

C. S. JARVIS,[27] M. AM. SOC. C. E. (by letter).[27a]—A simplified method of determining azimuth by direct observation of the sun has been ably presented by the author, but simpler methods of determining true bearings of a line deserve consideration and extended use.

It is possible to determine the true meridian within an error of 1′ or 2′ consistently day after day, by using methods similar to the one described by the author, differing only in details, with or without the solar attachment for the ordinary enginer's transit; but it happens too often that

[27] Hydr. Engr., SCS, Washington, D. C.

[27a] Received by the Secretary February 6, 1937.

the uncertainties or errors of bearing from such methods are considerably larger. No doubt this fact guided the standard practice of the General Land Office in requiring that the bearings of land-subdivisional lines be checked occasionally by observations on Polaris, and that the records of such tests and results be incorporated in the official field notes of public land surveys.

In view of the accessibility of correct standard time to within a second or two, through telegraphy and radio broadcasts, it appears that the transit of the sun's center across the meridian at apparent noon should be included. True solar time, derived from standard time by corrections for longitude and for the equation of time as published in a solar ephemeris, might be as much as 45 min slower or faster than standard time. An error of 4 sec in time would account for an error of 1′ of arc.

As the sun's semi-diameter varies from 15′ 45″ on July 1 to 16′ 18″ on January 1, and the time of its passing the meridian varies from 64 sec to 71 sec, as obtained from a solar ephemeris, it is quite feasible to observe the western limb of the sun at the corresponding time before apparent noon, and the eastern limb at the same interval after noon, but with the telescope reversed. These two lines should coincide on the meridian, except as affected by instrumental errors. A bisector of the small angle between them, therefore, should be the true meridian, obtained by the simplest method available, but susceptible to considerable errors if the time is not known accurately. However, by observing the sun's passage over the highest point of its orbit, together with its position a few minutes before and after noon (that is, at equal altitudes) both the apparent solar time and the meridian may be determined approximately, perhaps within 5′ as to bearing of the line.

Of all methods used by the writer for determining a meridian, the hour-angle observation of Polaris proved to be the most satisfactory. Utilizing twilight for illuminating the cross-hairs of the telescope, morning or evening, or both, the direct and reversed readings on the star and on the traverse line may be completed with very little time devoted to instrumental work and final calculations. Moreover, by occupying a station on the traverse line and sighting to one of the flags or marks, the observer is enabled to obtain the data for determining the bearings without the use of a flagman or rodman. As compared with observations at either culmination or elongation of Polaris, often involving tedious hours of waiting during the night, the hour-angle method has many advantages.

At the time of the vernal equinox (about March 21) the right ascension or time of meridian passage of Polaris at upper culmination is 1 hr 38 min by both sidereal and solar time, and the former gains nearly 4 min per day on the latter so as to accommodate one more day each year. It is evident, therefore, that on April 21 the upper culmination will occur 2 hr earlier, or at 11:38 A. M., and the lower culmination at 5:36 P. M., so that a twilight observation at about an hour later (assuming the latitude as 40° N), would define a line bearing 21′ east of north, and altitude of the star would be about 39 degrees. An error in time amounting to 3 min would account for an error of 1′ in azimuth at or near culmination; or for the entire hour

extending equally before and after elongation, the change of bearing would not exceed 1′, the actual azimuths at 30-min intervals being 80′.1, 80′.7, and 79′.8, beginning with 5 hr 30 min as the hour angle.

When the atmosphere is exceptionally clear and free from haze, dust, and smoke, it is quite possible to locate Polaris through a transit telescope a few minutes before sunset or after sunrise, and thus to complete the observation and calculations within daylight. In order to bring the star within the field of the telescope, it is necessary to set off an angle on the vertical circle to elevate the telescope equal to the latitude of the station plus or minus the correction for position of Polaris above or below the pole; or the cosine of the hour angle times 62′. Then, by setting the horizontal circle at approximately the correct position for the star, and using the tangent screw to search back and forth, the image should be made to streak part way across the field; and after centering it, the observation is soon completed.

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DISCUSSIONS

ANALYSIS OF CONTINUOUS FRAMES BY BALANCING ANGLE CHANGES

Discussion

BY MESSRS. L. J. MENSCH, MARVIN A. GRAY, AND A. FLORIS

L. J. MENSCH,[19] M. AM. SOC. C. E. (by letter).[19a]—The analysis of highly hyperstatic structures has not been taught in engineering schools or textbooks in such a practical manner that it can be used by busy engineers. They have avoided such structures, but when compelled to use them, have made safe guesses as to the stresses and, as a rule, have not produced economical designs.

Structures only one degree indeterminate are comparatively easy to solve, although requiring considerably more effort than ordinary beam construction and even such cases are avoided by many. Structures indeterminate in the second degree give twice as much trouble; and, three-degree indeterminacy necessitates much more mathematical skill and can only be designed by experts in this line.

Arched bridges are at least three times indeterminate when no hinges are used, evolve the expenditure of sizeable sums in construction, and have been analyzed regularly in the last twenty years by the classical theoretical methods. A continuous frame may be many times indeterminate, may evolve the expenditure of only moderate sums, and no busy engineer can afford the time for a complete theoretical analysis as taught in standard literature.

There is a crying demand for shorter and reasonably correct methods of design and Professor Grinter has tried to supply this deficiency by some novel procedures. The writer considers this and similar methods a retrograde step in structural engineering. The author makes the bold and gratituitous assumption that no side-sway may occur, or that it can easily be considered afterward. In the writer's opinion the analysis for side-sway is a much more difficult task than the analysis for vertical deformation only, and special short-cuts must be found before these methods are of much value to the profession.

NOTE.—The paper by L. E. Grinter, Assoc. M. Am. Soc. C. E., was published in September, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1936, by Messrs. John E. Goldberg, G. A. Maney, Paul Andersen, and William F. Luce; and in January, 1937, by Ralph E. Byrne, Jr., Jun. Am. Soc. C. E.

[19] Civ. Engr. and Constructor, Chicago, Ill.

[19a] Received by the Secretary January 8, 1937.

Consider, for example, the case of a railroad trestle with expansion joints over every third or fourth column. Side-sway may occur there. What prevents side-sway in a school building with classrooms on both sides of a comparatively narrow corridor?

The first example in this paper is a flagrant case of disregard of the fundamental condition of equilibrium, namely, that the sum of the forces acting in any of three principal directions must be zero. They are not zero in a horizontal direction by 25%, as shown by the analysis. Would any engineer be satisfied with a solution in which the sum of the computed vertical reactions would differ from the acting vertical loads by 25 per cent? By taking this unwarrantable liberty with statics, the author claims that he has found a new and revolutionary short-cut in the analysis of indeterminate structures.

There are cases in which this and related methods may be used. Symmetrical structures with symmetrical loading have no side-sway. If the side-sway is in such a direction that the structure leans against a rigid body, and temperature and shrinkage may be neglected, side-sway has no chance to act. In skeleton constructions, with many bents, the side-sway may be very small. There is no side-sway in the classical problem of Clapeyron for continuous girders.

In all other cases these methods are only crude guesses at solutions and require more brain work than the classical methods, provided St. Venant's principle is used, namely, that members in a continuous frame which are several spans or stories distant from the girder which is being analyzed have only a very small influence on the necessary elastic equations, as the following demonstration will show: Let AB be a girder under a load forming part of a frame, as shown in Fig. 19; let K, K'_A, K''_A * * *, be the ratio, $\frac{I}{L}$, of the various members; let M_A, M_B be the negative moments acting in $A\,B$,

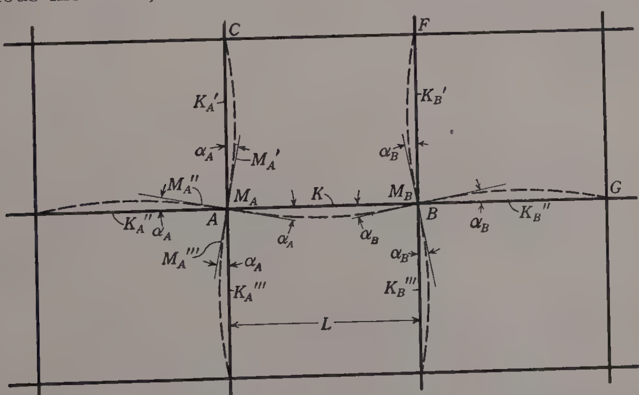

FIG. 19.

at the ends, A and B, according to the classical sign convention; and, let α_A, and α_B, be the rotations of the joints, A and B, due to the load and the restraint of the adjoining and further members. When these joints are

assumed to be rigid, it is clear that this rotation must be equal to the slope of the elastic line of all members meeting at any joint.

It is known, and may be found in almost any elementary textbook, that,

$$\alpha_A = \phi_A - \frac{(2\,M_A + M_B)\,L}{6\,E\,I} \quad \text{.................. (23}a\text{)}$$

and,

$$\alpha_B = \phi_B - \frac{(2\,M_B + M_A)\,L}{6\,E\,I} \quad \text{.................. (23}b\text{)}$$

For a uniform load the author gives $\phi_A = \phi_B = \frac{W\,L^3}{24\,E\,I}$; and for a concentrated load in the center, $\phi_A = \phi_B = \frac{P\,L^2}{16\,E\,I}$.

If α_A and α_B are known,, the unknown moments, M_A and M_B, can be obtained from Equations (23). The problem is reduced, to as close an estimate as possible, of the values of α_A and α_B from the deformation of the members adjacent to the joints, A and B, and the load on AB.

It is known from mechanics that for a member, AC, for example,

$$\alpha_A = \frac{M'_A}{m'\,E\,K'_A} \quad \text{........................ (24)}$$

in which m' is a factor depending on the restraint at the end, C. When C is hinged, $m' = 3$; when fixed, $m' = 4$; when AC is deformed so that the elastic line has a point of contraflexure just at mid-length of the member, $m' = 6$; and when AC is deformed so that the elastic line is symmetrical about the mid-point, $m' = 2$. In a previous discussion on this subject[20] the writer has shown that the final results for the unknown bending moments, M_A and M_B, do not greatly differ whatever the restraint is at the far end of the members adjacent to the joint, A, and for the case where this restraint cannot be readily estimated and may be assumed to lie between hinged and fixed conditions, a practical guess is:

$$\alpha_A = \frac{M'_A}{3.6\,E\,K'_A} \quad \text{........................ (25}a\text{)}$$

In rare cases the more precise value,

$$\alpha_A = \frac{M'_A}{E\,K'_A}\,\frac{1 + 4\,N_C}{4\,(1 + 3\,N_C)} \quad \text{.................... (25}b\text{)}$$

may be used. One can write:

$$\alpha_A = \frac{M'_A}{m'\,E\,K'_A} = \frac{M''_A}{m''\,E\,K''_A}$$

$$= \frac{M'''_A}{m'''\,E\,K'''_A} = \frac{M'_A + M''_A + M'''_A}{\Sigma\,m\,E\,K_A} = \frac{M_A}{\Sigma\,m\,E\,K_A} \quad \text{..... (26}a\text{)}$$

[20] *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1682.

and, similarly,

$$\alpha_B = \frac{M_B}{\Sigma\, m\, E\, K_B} \quad \text{.......................(26}b\text{)}$$

Let,

$$N_A = \frac{3\,K}{\Sigma\, m\, K_A} \quad \text{.......................(27}a\text{)}$$

and,

$$N_B = \frac{3\,K}{\Sigma\, m\, K_B} \quad \text{.......................(27}b\text{)}$$

then, by combining Equations (26) with Equations (23) one easily obtains for a uniform load:

$$M_A = \frac{w\,L^2}{4}\;\frac{1 + 2\,N_B}{4\,(1 + N_A)\,(1 + N_B) - 1} \quad \text{............(28}a\text{)}$$

and,

$$M_B = \frac{w\,L^2}{4}\;\frac{1 + 2\,N_A}{4\,(1 + N_A)\,(1 + N_B) - 1} \quad \text{............(28}b\text{)}$$

For a concentrated load in the center, $\frac{w\,L^2}{4}$ must be replaced by $\frac{3\,P\,L}{8}$.

Once the moments, M_A and M_B, are computed they can be distributed to the adjoining members, AC, for example, as:

$$M'_A = M_A\,\frac{m'\,K'_A}{\Sigma\, m\, K_A} \quad \text{.......................(29)}$$

The moment, M'_A, produces a moment at the far end of AC, depending on the value of m', as follows: When $m' = 2, 3, 3.6, 4, 6,$ or $4\,\frac{(1 + 3\,N_C)}{(1 + 4\,N_C)}$,

$$M_{CA} = M'_A\left(-1, 0, \frac{1}{3}, \frac{1}{2}, 1, \frac{1}{2 + 6\,N_C}\right) \quad \text{..........(30)}$$

The moment, M_{CA}, can again be distributed to the adjoining members meeting at C, if found desirable; but this is generally unnecessary. In most cases it is near enough to compute the maximum positive moment in AB as the center moment by,

$$M = \frac{w\,L^2}{8} - \frac{M_A + M_B}{2} \quad \text{...................(31)}$$

If it is desired to compute the maximum negative moment in AB at B, one must assume also that the girder, BG, is loaded, and compute from Equations (28) the negative left end moment, and then by distribution according to stiffness, as shown in Equation (29), the moment, M_{BA}, which has to be added to the moment, M_B, previously found.

This is the classical method of moment distribution, except that the distant spans have so little influence, in most cases, on the elastic equations of the particular member, which is being analyzed, that they may be neglected in practice. This holds true whether Castigliano's theorem, the Maxwell equations, or the principle of virtual velocities are used. These are only different names for the identical elastic equations.

Experts in the analysis of hyperstatic structures of the continuous-frame type have rarely used the latter principles and have found that the moment-area principle, in combination with the rotation of the joints, offers a number of short-cuts, not inherent in the foregoing methods, as may be found in the excellent works of Professor E. Winkler[21], the works of W. Ritter[22], W. Gehler[23], A. Strassner[24]; etc.

By the method herein presented by the writer an attempt will be made to analyze the structure shown in Fig. 2. All the K-values are shown in that diagram and $m = 3$, except for DC where $m = 4$.

From Equations (27), for Beam BD: $N_B = \dfrac{3 \times 4}{3 \times 2} = 2$; and $N_D = \dfrac{3 \times 4}{3 \times 2 + 3 \times 3 + 4 \times 3} = 0.445$; and, from Equations (28): $M_B = 0.0289\ w\,L^2$; and $M_D = 0.0765\ w\,L^2$.

It is clear that the moment, M_{BA}, at B is also $0.0289\ w\ L^2$; and the horizontal reaction at $A = 0.0289\ w\ L$. The author did not give the inclination of the member, BA, as implying no importance in his opinion, and one may assume the member to be vertical. The moment, M_D, must be distributed in the proportion, $3 \times 2 : 3 \times 3 : 4 \times 3$, into the members, DE, DF, and DC, respectively, and these moments are easily found to be 0.017, 0.0255, and $0.034\ w\ L^2$, which lead to reactions of $0.017\ w\ L$, $0.0153\ w\ L$, and $0.0567\ w\ L$, at Points E, F, and C, respectively.

The horizontal reactions at Points A and E are acting to the right and their sum is $(0.0289 + 0.017)\ w\ L$, whereas the horizontal reaction at Point C is acting to the left and is only $0.0567\ w\ L$—or the opposing reactions to the latter are more than 24% greater, an impossible condition. Sidesway will occur in the girders to the left so that the moment, M_B, is increased about 20% and the moment, M_D, decreased a similar amount.

The girder, DF, may be analyzed as follows: The stiffness of the member, DF, beyond the hinge, F, being zero, as there is no restraint to the right, $N_F = \infty$. From Equations (28),

$$M_D = \frac{3\,P\,L}{8}\;\frac{1 + 2\,N_F}{4\,(1 + N_D)\,(1 + N_F) - 1} \quad \ldots\ldots\ldots\ldots (32)$$

Dividing the numerator and the denominator by the infinite value of N_F:

$$M_D = \frac{3\,P\,L}{8}\;\frac{2}{4\,(1 + N_D)} = \frac{3\,P\,L}{16\,(1 + N_D)} \quad \ldots\ldots\ldots\ldots (33)$$

[21] "Vorträge über Brückenbau", G. Gerold, Vienna, 1875.
[22] "Graphische Statik", III, Luric, 1900.
[23] "Der Rahmen", Wm. Ernst & Sohn, Berlin, 1913.
[24] "Neuere Methoden", Berlin, 1916.

in which, $N_D = \dfrac{3 \times 3}{3 \times 2 + 4 \times 3 + 3.6 \times 4} = 0.278$; and $M_D = 0.148\ P\ L = 4\,930$ ft-lb. This negative moment will be diminished by the cantilever moment at F of 20 000 ft-lb, as follows:

From Equation (25b),

$$m = \frac{4\,(1 + 3\,N_D)}{(1 + 4\,N_D)} = \frac{4\,(1 + 3 \times 0.278)}{1 + 4 \times 0.278} = 3.46$$

From Equation (30)

$$M_{DF} = \frac{20\,000}{2 + 6 \times 0.278} = 5\,460 \text{ ft-lb.}$$

which is a positive contribution to the moment in DF at D, producing there a positive moment of $5\,460 - 4\,930 = 530$ ft-lb; which must be distributed according to the stiffness ratios of $3 \times 2 : 4 \times 3 : 3.6 \times 4$ to the members, DE, DC, and DB, respectively. These additional moments will decrease somewhat the sum of the horizontal reactions found for the uniform load on BD alone, but will still leave a serious difference.

The writer claims that Equations (28) are so simple that they can be used by busy engineers and will give reasonably correct results in a few minutes without deep thinking and less likelihood of mistakes than the new methods, such as that proposed in this paper. He would advise the author to improve his scholarly paper by showing more practical methods of estimating the influence of side-sway, which is of greater importance at present than the computation of the primary moments. There are plenty of methods for finding primary moments in engineering literature, whereas side-sway has generally been overlooked, or, if treated at all, unbalancing of the vertical reactions has often resulted.

Marvin A. Gray[25] Esq. (by letter).[25a]—The purpose of the following discussion is to test the theory of this paper in all its technical phases. Throughout, Professor Grinter uses such terms as "slope" and "deflection", apparently without realizing that it is, in fact, "slope deflection", becauses he uses ϕ in special cases instead of the more generally known symbol, θ, that is used in connection with slope deflection. He presents some values of ϕ derived by the conjugate-beam method (a variation of moment areas) and each of these ϕ-values needs a separate derivation. This type of method is meant to displace slope deflection, which needs only one proof or derivation and the single set of unknowns (θ-values) instead of θ-values and ϕ-values.

To show how closely this paper follows the slope deflection method the writer herein develops the ϕ-values by using the slope deflection equation,

$$M_{AB} = M_{F\text{-}AB} - \frac{2\,E\,I}{L}\,(2\,\theta_A + \theta_B) \quad \ldots\ldots\ldots\ldots\ldots (34)$$

[25] Chicago, Ill.

[25a] Received by the Secretary January 29, 1937.

in a simple beam. For a concentrated load at mid-span, $-\theta_B = \theta_A$; so that (with $M_{AB} = 0$) Equation (34) yields for θ_A (or for ϕ_A and $-\phi_B$): $0 = \frac{PL}{8} - \frac{2EI}{L}(2\theta_A + \theta_B)$; or,

$$\theta_A = \frac{PL^2}{16EI} = -\theta_B \quad\text{.....................(35)}$$

For a uniform load on the entire span, θ_A (or ϕ_A and $-\phi_B$) equals $\theta_A = \frac{wL^2}{12}\,\frac{L}{2EI} = -\theta_B$, that is:

$$\theta_A = \frac{wL^3}{24EI} \quad\text{........................(36)}$$

For a concentrated load not at mid-span, in a similar manner, Equation (34) yields, for θ_A (or ϕ_A);

$$\theta_A = \frac{Pab^2}{L^2}\,\frac{L}{2EI}\left(\frac{2}{3} + \frac{a}{3b}\right) = \frac{Pab}{2EIL}\left(\frac{2b}{3} + \frac{a}{3}\right) \quad\text{....(37a)}$$

and, for θ_B (or ϕ_B):

$$\theta_B = -\frac{Pa^2b}{L^2}\,\frac{L}{2EI}\left(\frac{2}{3} + \frac{b}{3a}\right) = -\frac{Pab}{2EIL}\left(\frac{2a}{3} + \frac{b}{3}\right) \quad\text{..(37b)}$$

For an induced moment, M, at End A, with $\theta_A + 2\theta_A = 0$; and $\theta_A = -2\theta_B$, Equation (34) yields, for θ_A (or ϕ_A):

$$\theta_A = \frac{L}{2EI}\,\frac{2M}{3} = \frac{ML}{3EI} \quad\text{.................(38a)}$$

and, for θ_B (or ϕ_B):

$$\theta_B = -\frac{\theta_A}{2} = -\frac{M}{6EI} \quad\text{..................(38b)}$$

and if the result appears with the wrong sign, the author's sign reversal explains that the author has taken reaction instead of action. The writer has now proved definitely that the ϕ-value is a special value or solution obtained directly by slope deflection from the value of θ.

Having stated his thesis, the author next proceeds through part of the paper to give a proof for the moment distribution method, and succeeds in demonstrating that it is really based on, or derived from, slope deflection; as, for example: "Accordingly, it follows that the carry-over factor [θ_A or θ_B] for angle change is of negative sign, namely, -0.5". * * * "All of the carry-over angles to the far ends of the other members meeting at the joint are simply -0.5θ", etc. The balancing process of the slope deflection method (which is a part of all similar "methods") is based on the following equations:

$$M_{AB} = M_{F\text{-}AB} - \left(\frac{2EI}{L}\right)(2\theta_A + \theta_B) \quad\text{.............(39a)}$$

and,

$$M_{BA} = M_{F\text{-}BA} - K\,(2\,\theta_B + \theta_A) \quad \text{.........(39b)}$$

The balancing part of Equation (39a) is $= \left(\frac{2\,E\,I}{L}\right)(2\,\theta_A + \theta_B)$; and that of Equation (39b) is $-\ K(2\theta_B + \theta_A)$. If, momentarily, the direct effects of M_{AB} and $M_{F\text{-}AB}$, are disregarded, the following result is obtained due to M_{BA}: $0 = 0 - K\ (2\,\theta_A + \theta_B)$. Then, $\theta_A = -\ 0.5\,\theta_B$, or,

$$K\theta_A = -\ K\,0\,5\,\theta_B \quad \text{.........(40)}$$

In the moment distribution, $K\ (2\ \theta_A)$ is solved for directly and $K(2\ \theta_B)$ in Equation (39b) is $-\ 0.5\ (K\ 2\ \theta_A)$, as in Equation (40). In other words by slope deflection, the writer has proved in a truly direct method, that an angle at one end, A, of a beam induces an angle (or its equivalent moment) at the other end, B, opposite in sign and one-half its value; or θ_A induces a value, $-\ 0.5\ \ \theta_A$ (or $-\ 0.5\ K\ \theta_A$), at Point (B), the far end of the beam; and, conversely, Angle θ_B, from an external moment, produces a rotation (or a moment), $-0.5\theta_B$ (or $-0.5\ K\ \theta_B$), at End (A) the rear end of the beam, which is what the author terms the rotation "balancing angle". Expressed in terms of moment distribution multiplying by K ($K\ \theta$ is a moment), a moment induced at one end creates a moment (or angle) at the other end equal to $-\ 0.5$ of the original induced moment (or angle), the correction or carry-over factor that is used in the moment distribution method. Thus are the foregoing quotations from the paper explained; and the contention that moment distribution (of which the method of balancing angle changes is a branch) is a variation of slope deflection by changing symbols, is proved conclusively.

The subject of continuity at joints is by far the most important phase of rigid frames as far as practice is concerned, because without this knowledge, all the theory pertaining to it does little good. However, if in a homogeneous beam it works on the basis of plastic flow as the author infers, the entire beam would deform until it acts like a cable. It is more probable that one of two conditions may exist: (1) The member may be free to move due to loose rivets, or there may be "play" in the connection; or (2), rivets or the connection may yield because they are overstressed until the ends finally hold or balance the load. The first case seems illogical, and failure of the material is very probable in the second case; but in either case a definite θ-value can be given for the movement, with a solution by slope deflection.

The publication of this paper affords an opportunity to clarify the relation between rigid frame methods (solutions) and their basis, the slope deflection method. The author is correct in emphasizing the superior virtue of angle movement, or the balancing of angle changes; such advantage has been shown many times in papers on slope deflection.[26] The adaptation of the standard Slope

[26] See, for example, *Engineering News-Record*, Vol. 107, No. 20, p. 770, November 12, 1934.

Deflection Equation[27] constitutes the most accurate, as well as the fastest, method having a theoretical proof based on angle movement. Slope deflection and its method of solution by converging approximations will be of great value in the new fields of structural analysis, such as airplanes, airships, light-weight trains, and high-strength large bridges.

A. Floris,[28] Esq. (by letter).[28a]—The analysis presented in this paper is a complicated variation of the well known and widely used simple method of moment distribution.

As a base system the author takes a beam that is freely supported or free to rotate at the ends. This is one of the limiting cases. The other limiting case, the fully restrained beam, used in the moment distribution method[2] could not be utilized as a base system in the author's analysis, because the tangent to the elastic curve at the supports being horizontal, there are no angles to distribute.

However, the advantage of the choice of the angle changes instead of the fixed end moments is debatable. In general, engineers are not concerned with angle changes of bars framing into a joint. It is true that in the slope-deflection method these changes are introduced in the analysis, but this is done for the purpose of reducing the degree of statical indeterminacy of the structure. The procedure of balancing the statically indeterminate quantities is the same in the author's method as in the moment-distribution method. Hence, there is no need to use the less convenient angles, if the required moments can be found more directly by moment distribution.

In the author's analysis, the angle changes at a joint must be distributed to all members framing into it, in such a way, that, by changing the slopes, the continuity of the structure is restored. By doing this, however, moments are induced at the ends of these bars. In the moment distribution method the moments produce the angle changes. In the author's analysis the angle changes produce the moments. Consequently, the angle changes at the ends of bars are inversely, and not directly, proportional to the K-values, as the author states. A simple analysis will prove it. In spite of this discrepancy, the results obtained are correct because of the fact that the distribution is made by taking percentages of the stiffness of the bars framing into a joint.

With the use of the base system in the moment distribution as well as in the angle distribution, methods, the continuity of the structure is destroyed. To restore the continuity by unlocking or restraining the joints, the moments or angle changes must be distributed to the members framing into a joint in proportion to their capacity to resist such moments or angle changes. This is the balancing process which, in turn, influences the opposite end of the bars under consideration. This influence upon the opposite end of the bars is the carry-over process. The moment distribution method is based upon these two simple steps.

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[27] "Engineering Studies," *Bulletin No. 1*, Univ. of Minnesota, March, 1915.
[28] Dipl.-Ing., Los Angeles, Calif.
[28a] Received by the Secretary December 21, 1936.
[2] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1.

The author's explanations regarding the application of his method to side-sway produced by lateral or unsymmetrical loading are extremely vague. In this case, the freely rotating bar at both ends cannot be used as a base system throughout the structure. The insertion of hinges in all joints necessitated by the choice of this beam produces a system that does not possess lateral stability. Perhaps the frames with columns fixed at the base, connected to beams that are hinged at the ends, can be chosen as a base system. The interesting properties of these frames, which are stable against lateral forces, have been discussed by the writer elsewhere.[29]

[29] "Types of Rigid Frames for Existing Buildings to Resist Shocks", by A. Floris. *Southwest Builder and Contractor*, July 13, 1934, p. 22.

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DISCUSSIONS

THE MODERN EXPRESS HIGHWAY

Discussion

BY MESSRS. HENRY B. ALVORD, ROBERT EUGENE HILES, AND CHANDLER DAVIS

HENRY B. ALVORD,[40] ASSOC. M. AM. SOC. C. E. (by letter).[40a]—If for no other reason than that it brings to a focus the relation of considerations of safety to the design of a highway, this paper is a timely one. Safety should be a dominant principle in the design of through, or express, highways as it already has been accepted in the design of other structures, such as bridges, buildings, and dams.

In order that this problem may be further particularized the writer presents herewith, a tentative set of suggestions in outline form which would apply to the design of an express highway. The attempt is to present a complete yet concise summary that will include all features necessary in a safe design. The writer may have omitted some points or given incorrect values in some instances; but if ideas such as these could be formulated and studied, they might help toward an agreement as to what is, and what is not, essential in such a program.

Competent engineers consider the following items to be necessary features of a safely designed through-traffic artery or express highway:

(1) Maximum allowable speed, 100 miles per hr;

(2) Two roadways of three lanes each, separated by a center strip having a width of 20 ft and a 20° sloping curb, each roadway to consist of two 12-ft lanes, the surface of the outside 20-ft parking lane to be of rough texture;

(3) Minimum radius of horizontal curve, 3 000 ft;

(4) Transition (spiral) curves;

(5) Horizontal curves to be super-elevated for speed of 60 miles per hr;

NOTE.—The paper by Charles M. Noble, Assoc. M. Am. Soc. C. E., was published in September, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1936, by Messrs. Fred Lavis, Joseph Barnett, G. E. Hawthorn, John F. Fairchild, Leslie R. Schureman, and C. H. Purcell; December, 1936, by Messrs. Elmer R. Haile, Jr., H. W. Griffin, and T. T. Wiley; January, 1937, by Messrs. F. L. McRee, Theron M. Ripley, W. W. Crosby, Richard S. Kirby, Harold M. Lewis, George Conrad Diehl, and William E. Rudolph; and February, 1937, by Messrs. A. C. Dennis, and J. C. Carpenter.

[40] Prof., Civ. Eng., and in Chg. of Dept., Northeastern Univ., Boston, Mass.

[40a] Received by the Secretary January 28, 1937.

(6) Maximum grade, 5%;

(7) Minimum vertical curve length = 160 times the percentage change in rate of grade. (This corresponds to a tangent offset of $\frac{5}{16}$ ft, 100 ft from any point of tangency);

(8) Minimum profile tangent, 1 500 ft;

(9) Minimum sight distance, 800 ft;

(10) Minimum horizontal tangent, 1 000 ft;

(11) Overpasses for all important cross-traffic. (No crossings of traffic at grade);

(12) Restriction of all side entrances to a minimum spacing of 5 miles. (No break in center strip at side entrances nor within 1 000 ft of a side entrance, and all breaks in center strip protected by a deceleration and acceleration lane of 500 ft);

(13) Acceleration and deceleration lanes at all side entrances (minimum length, 500 ft);

(14) White stripes separating lanes to control passing;

(15) Adequate lighting to provide clear visibility of all objects in roadway for a maximum distance of 2 000 ft, such lighting to be entirely in addition to, and independent of, automobile headlights;

(16) Surface textures to provide at least a coefficient of friction of 0.5; smoothness of profile to be consistent with operation at 100 miles per hr;

(17) Surface of extreme right-hand lane of extra rough texture and to be used for emergency stops only; all other lanes to have specified minimum speed limits;

(18) No entrance or egress permitted from roadways to and from adjacent land; all use of marginal land for commercial purposes to be eliminated by the non-access feature herein contained;

(19) Utility wires to be placed in conduits or on poles adjacent to right-of-way boundary lines;

(20) Directional signs restricted to a few words, placed sufficiently in advance of the point to which they refer, and to be of such size of type as to be easily legible by an observer passing at a rate of 100 miles per hr; and,

(21) Any installation of traffic lights of any type at any place in this express highway is clearly incompatible with proper design as indicated heretofore.

Any set of numerical requirements such as Items (1) to (21) probably will not be consistent for use in the entire United States. At least three classifications are necessary: One for a climate where frost and ice never cause a serious problem; one where ice and snow are common during the winter months; and one where the topography is exceptionally rough.

The Civil Engineering Profession is charged with the technical responsibility inherent in highway design. It is evident that the demands of highway traffic have grown (like "Topsy") out of all bounds. These demands have swamped the highway designing engineer. Either his recommendations often have been inadequate, or his design has been emasculated by his superiors under the guise of economy, falsely so-called.

If a structural engineer allows a design of a building to pass his desk when he knows it is unsafe, he is not commended for the economy of the design. Safety properly takes precedence over economy. Safety principles accepted in building design apply equally to highway design, namely, safe capacity and safe use. If this premise is correct, the conclusion is obvious—an inadequate design of a highway from the standpoint of safety is unprofessional.

Robert Eugene Hiles,[41] Assoc. M. Am. Soc. C. E. (by letter).[41a]—In the paper by Mr. Noble, attention is called to the distinctive need for an inventory of the present worth of the modern express highways, as regards the purposes for which they were built, and the additional services that they are now rendering. He has suggested tests, research projects, and methods of handling the present and future requirements of heavy traffic conveniently and safely.

Briefly, the writer proposes that the modern express highway should be built and operated in a manner similar to the present modern express railroads, which have most certainly handled their large volume of freight and passenger traffic safely and conveniently for the public welfare.

In introducing this discussion, it might be interesting to call attention to the astonishing fact that during the same year (1935) that Mr. Noble used to show that there were 36 100 deaths due to automobile accidents, there was not a single death of a paid customer due to accident on the railroads. No other test than that should be needed, therefore, to prove that the system of turning 50 000 000 vehicles, more or less, loose on an open highway with just any one operating and guiding them at any rate of speed, perhaps to 120 miles per hr, proved to be wrong in the 36 100 cases in 1935. It would also be just as wrong to put a steering wheel on a modern stream-line train and turn it loose on the open highway at 100 miles per hr. The modern highways handle much more traffic than the railroads; so why not control them as rigidly as the railroads? No recommendations are made herein to handle local traffic, except that which is permitted entrance at established express terminals after complying with and passing certain rigid regulations and inspections for road-worthiness, safety, etc. The construction of these express highways and facilities would require the building of new roadways where they could not follow the alignment and otherwise utilize the present highways. The adequacy of the present system of handling the express highway will certainly and surely be tested in time of war when man power will be scarce to guide all the machines of war besides all the vehicles for domestic and military transportation. Finally, the payment for this proposed project would not only be made from tolls charged on vehicles using the express highway and rents charged for concessions in the terminals, etc., but also by reason of the savings accrued from reducing the present type of highway pavement to that actually required per lane width of the proposed express highway track.

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[41] Res. Engr. Insp., P.W.A., Dover, Ark.

[41a] Received by the Secretary January 27, 1937.

Express-Highway Tracks.—Why not put the modern express vehicles on tracks or in guides so that the personal element of guiding will be eliminated from the many duties of the driver? The design of these tracks should be effective and of sufficient strength to guide the standard design trucks and truck trains without the aid of a driver in the vehicle and should give due consideration to the following factors: Installation, replacement, and adaptability to present highway slabs and bridges; adaptability of present vehicles to a standard express vehicle and track gage; selection of materials for guides; hazards of the elements; alignment and right-of-way clearances; automatic control devices with particular attention to the photo-electric cells and other safety devices for warnings and control under all hazardous conditions and locations along the right of way; design of track appurtenances, such as switches, frogs, cross-overs, sidings, passing tracks, and terminal facilities; and, finally, the adoption of a guide that will permit the use of the present type and design of automobiles, so that with slight delay local traffic, upon reaching an express terminal, may enter and be permitted the better facilities of safety and convenience afforded by the modern express highway system.

Physical Characteristics.—As shown by the record of accidents in Chicago, Ill., where certain sections of highway were increased from normal to 100 ft in roadway widths, the number of accidents increased instead of decreased. Apparently, then, this is not the solution, but rather the separation or division into lanes by mechanical means.

The proposed double-track express highway would handle, safely and conveniently (if properly regulated), as much as the present highways handle in perhaps four lanes. Passing tracks and sidings would be required, however, for cases of emergencies and convenience at various intervals along the system.

Any speed could be maintained either by law or by governors on the vehicles which could be checked at the terminals from previous tickets showing previous records of tests, "roadability" performance, accidents with rating as to frequency and seriousness, etc. By maintaining certain legal speeds the necessity and hazards of passing vehicles are eliminated.

Truck and passenger trains controlled by the express highway system, or privately owned passenger and freight train and single vehicles, could be operated over the same system.

Design.—Photo-electric cells, lights, neon tubing, and other control equipment could be placed on the front and rear of all vehicles and at certain necessary points along the right of way to maintain certain desired distances between vehicles and otherwise to control the speed and operation of the moving vehicles.

Certain familiar design assumptions (such as the increase in load or eccentric loading to trusses and girders, due to the shifting about of the design load from one curb to another, in order to produce probable maximum design conditions on the present type of bridge floor) would be eliminated by the proposed track system and in many structures this would affect

a material saving in cost. Furthermore, the entire road-bed would not have to be paved. This advantage, however, would depend on the type of tracks or guides adopted.

Control.—Vehicles that failed to meet the requirements of speed, safety, and "roadability", while using the express highway, would be fined or otherwise penalized or disbarred in accordance with the severity and frequency of the offense. The routing and train, or vehicle, movement would be controlled by express dispatchers. The design and construction, supervision, inspection, collection of tolls, rentals, fees, fines, etc., operation, and maintenance would be under the direction of the present State highway commissions.

Payment.—The basis for payment in the form of tolls or express highway charges would be in accordance with the weight per mile hauled over the roads, payable and collected in advance at the express highway terminals. As noted under the heading, "Control", rentals, or leases for concessions at the express terminals, together with fines and penalties collected, would apply on operation and maintenance costs. Any saving by reason of design, in favor of the proposed modern express highway system should be credited toward payment. However, the greatest saving anticipated by this system is not financial, but is the saving of human lives, which at the present rate is a loss of 100 per day; and, the ultimate goal of the modern express highway should be the almost perfect record of the modern express railroads.

Conclusion.—The recommendations and suggestions contained herein are submitted for consideration, as suggested by Mr. Noble, for: Design, construction, operation, test, and research, to determine a safe and convenient modern express highway; and, perhaps, to prove that this can be accomplished by eliminating, as much as possible, the personal element from the control and operation of modern express vehicles on the highways.

CHANDLER DAVIS,[42] M. AM. SOC. C. E. (by letter).[42a]—The modern express highway, as described by Mr. Noble, is undoubtedly the highway of the future. Whether it will be feasible to permit driving at 100 miles per hr is very doubtful.

For 1935, Table 1 records 826 690 accidents of all kinds on American streets and highways, of which 374 490 were collisions between automobiles (that is, 45.3% of the total number), resulting in killing 24.6% and injuring 50.3% of the total reported for the year—this large and appalling list in spite of the numerous laws and ordinances.

Suggestions for minimizing such accidents and making the high-speed roads safe is treated by the author, who states that a car traveling at 100 miles per hr requires 617 ft in which to stop, provided the car is fitted with four-wheel brakes. To this value is added 219 ft for a lag of 1.5 sec for an emergency stop; that is, a total of 836 ft is required if a car should be required to stop suddenly. Naturally, each vehicle on the road will require the same space for a factor of safety and, consequently, each mile of road is

[42] Cons. Engr., New York, N. Y.

[42a] Received by the Secretary February 4, 1937.

limited to seven cars, if a speed of 100 miles per hr is to be maintained. This seems an absurd conclusion. Where can one find the data which will prove that, for safety, the road spaces given by Equation (2) are correct and conform to the proper use of highways. The United States Army has made a thorough study of the problem of moving supplies by truck and in developing rules of the road for motorized trains have reached conclusions, which seem to be based on Equation (2), or on some similar formula. It must be borne in mind, however, that the military units do not, and are not permitted to, exceed a speed which has been worked out as safe, although at times in case of an emergency the highest possible speeds are permissible; the safe rate for moving long columns (military) has been limited to 12 to 18 miles per hr.

The Army unit is the motor company, consisting of fifty-four trucks. Each vehicle is allowed a road space of 30 yd, which permits a maximum safe speed of 40 miles per hr. This is approximately the maximum speed found to be permissible on present-day highways, although, generally, it is exceeded. The ordinary speed for the Army is fixed at 20 miles per hr, although light tanks, carried on their trucks, are moved at 40 miles per hr when required. At night, it may be necessary to march without lights and naturally under such conditions (even when the cars may use their lights) the speed is considerably reduced. As 50 yd is left free between each company, it will be seen that one company requires 1 mile of road when moving; or at speeds as great as 40 miles per hr, with vehicles maintaining the proper distances, each mile is limited to fifty-four cars.

Columns of motor-driven vehicles can be moved safely, day and night (the latter in case of an emergency only) at 40 miles per hr, provided all road crossings and junctions are properly controlled. Cars should only be allowed to enter the road under the direction of a traffic control; therefore, it seems that the approaches to the express highway should be similarly controlled. This would call for as few approaches as possible between terminals, and one for each lane so that there would be no crossing of cars at any time.

Undoubtedly, the perfect highway will be constructed eventually; but the writer believes that the high-speed commercial roadway is a long way from realization.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SELECTION OF MATERIALS FOR ROLLED-FILL EARTH DAMS

Discussion

By Messrs. C. H. Kadie, Jr., and Ralph Bennett

C. H. Kadie, Jr.,[31] Jun. Am. Soc. C. E. (by letter).[31a]—Valuable information on the selection of materials suitable for earth dams is presented by Mr. Lee. The author has developed a criterion for the selection of materials which will give excellent results within the stated limits. However, these limits seem to be too narrow. The writer believes that the design of the dam is predicated upon the available material, and that a structure may be designed and built to meet all necessary requirements by using materials which fall well outside the limits presented by Mr. Lee.

The design of an earth dam and the type of construction equipment selected to build it will vary with the materials used. To establish limits of satisfactory material would be to destroy one of the most attractive features of an earth dam, its absolute flexibility. Limiting the type of earth dam under discussion to rolled fill does not materially affect the limits of usable material. Having selected a borrow-pit, it is only necessary to design the structure and specify the construction equipment in order to obtain a thoroughly satisfactory dam. The shortest haul will not necessarily result in the most economical construction. The greatest economy will be found in the least number of station yards and the absence of any unusual construction equipment.

The material which will facilitate the construction of an economical earth dam lies within a small and known radius of the dam site. These materials should be investigated thoroughly for those desirable characteristics completely summarized by Mr. Lee under the heading "Requirements for Suitability". Materials much more harsh than those shown in Fig. 7 may be used by selecting a very heavy sheepsfoot roller. In fact, the coarse limit established

Note.—The paper by Charles H. Lee, M. Am. Soc. C. E., was published in September, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1936, by Messrs. T. T. Knappen, and Paul Baumann; January, 1937, by Messrs. William C. Hill, A. Floris, and Fred D. Pyle; and February, 1937, by Messrs. Joel B. Cox, Stanley M. Dore, John E. Field, William P. Creager, and Joseph Jacobs.

[31] Asst. Engr., U. S. Bureau of Reclamation, Yuma, Ariz.

[31a] Received by the Secretary January 18, 1937.

by Mr. Lee would probably require a roller weighing more than 2 000 lb per lin ft, which is somewhat heavier than the usual design.

As shown by Mr. Lee, Talbot's general grading formula, Equation (1), is very well suited as a criterion in selecting materials. The more nearly a material conforms in mechanical analysis to the curve established by this equation, the greater will be its compacted density. This will be found especially useful in making a determination of the depth of cut for the most desirable material in a stratified borrow-pit. It must be remembered, however, that the present power shovels are seldom able to mix materials satisfactorily during excavation, from a cut that is more than 20 ft deep. It is a fairly simple matter to prepare mechanical analysis curves of composite samples to various depths. By solving for n in Equation (1) for a series of points, the actual size of the screens used is to be preferred, the curve which most nearly corresponds to some one value of n may be determined. It will be found that many materials are quite erratic in the part of the curve referring to material larger than 1 in. If the percentage of material greater than this size is small, it will have little effect on the result and may be ignored. Having selected the best depth of cut for scattered test pits, the same system may be used in choosing the best pits.

Mr. Lee has shown very clearly the known soil characteristics which may be determined by test. A knowledge of these characteristics is essential in choosing the material for a rolled-fill earth dam. One characteristic only mentioned briefly by him has proved extremely useful in the control of moisture during construction. Quoting Mr. Lee (see "Compaction"):

> "Higher water content [above critical] results in increased plasticity, larger voids, and less density, until a point is reached at which equipment will not be supported. A lower content [below critical] may produce a fill of greater apparent stability with little or no plasticity, but one having a greater void volume and less density."

This is a relation always found in soil compaction. For any material the critical moisture is defined by the peak of the compaction curve. Fig. 20 shows a typical density-moisture (compaction) curve with a typical resistance to

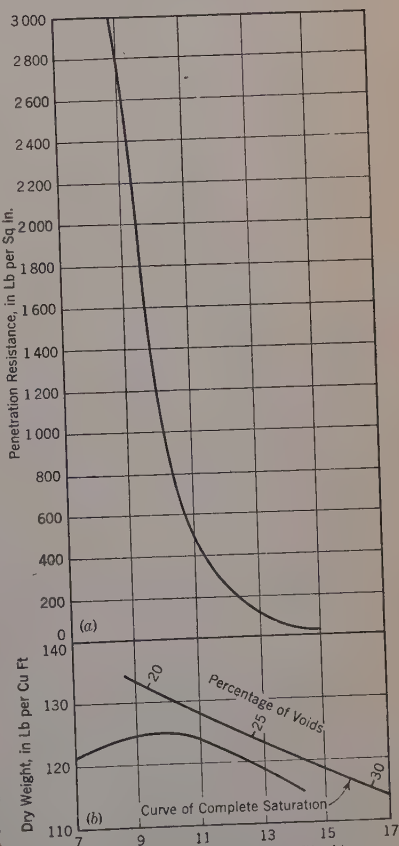

FIG. 20.

penetration curve (plasticity) measured in pounds per square inch. Since it is essential to obtain the correct moisture content, a knowledge of this relationship is necessary because it affords a convenient and rapid method of determining and controlling the moisture content. The details of making this test and the apparatus necessary have been presented in a series of papers by R. R. Proctor, M. Am. Soc. C. E.[32]

In the foregoing quotation the fact is mentioned that very high (above critical) moisture contents will result in a fill which will not support equipment. For satisfactory construction conditions the hauling units should not be so heavy that they will not be supported by the fill at a moisture content found to be critical for the rollers in use. Fig. 21 shows a series of compac-

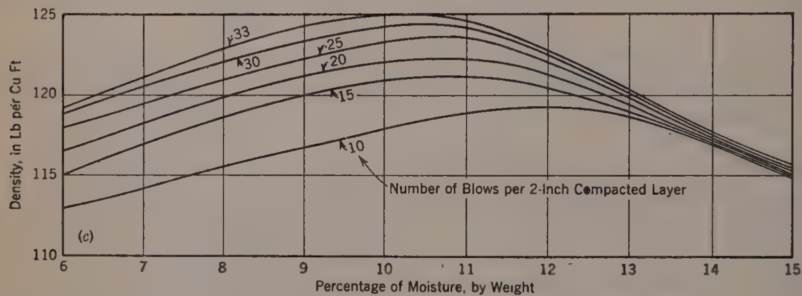

FIG. 21.—NUMBER OF BLOWS PER 2-INCH COMPACTED LAYER

tion curves developed from the same material at various degrees of work. As the work increases (measured in blows per 2-in. compacted layer delivered by dropping a 5.5-lb rod, 18 in.) the density increases and the critical moisture content decreases. The additional work can be obtained by increased rolling or, best, by imposing additional weight on the rollers. With the present tendency toward large hauling units, and the necessity of using matched equipment (rollers and trucks), it would be best to specify a very heavy roller. This would make it possible to construct the fill at a lower moisture content and would avoid a condition of either too much moisture to support the hauling equipment, or a too little moisture content for the compaction units.

RALPH BENNETT,[33] M. AM. SOC. C. E. (by letter).[33a]—In a discussion of earth dams it is usually assumed that only untreated materials are to be used. Raw fill is to be dug in convenient pits and consolidated by purely mechanical means. In other arts, there has developed a considerable use of earth which has been consolidated by the addition of imported media.

For sands the use of chemicals as a binder results in a concrete for which only a limited quantity of chemical is required. The result is an impervious, hard material made in place, and without requiring the elaborate and tedious handlings of materials necessary for the making of Portland cement concrete.

[32] *Engineering News-Record,* August 31, September 7, 21, and 28, 1933.
[33] Cons. Engr., Los Angeles, Calif.
[33a] Received by the Secretary January 14, 1937.

Although already extensively used to form underground water cut-offs and to prevent infiltration into deep excavations, there does not seem as yet to have been any chemical binder used to produce a strong and impervious face on dams, levees, or ditch banks.

For clays an extensive use of water-emulsion oil admixtures has resulted, during 1936, in the production of a water-proof material which will carry loads up to and in excess of those which the same clay, hard and dry but untreated, would carry. Clays when treated with the emulsion rolled in place, and allowed to dry out, do not again absorb moisture. They are permanently cemented by the microscopic oil film which has replaced the water film that originally held the particles. The quantity of emulsion used is determined by the extent of the fines in the raw material as the binder acts only on them. The total percentage on ordinary clays is very low and must not be exceeded, or it will lose effectiveness.

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DISCUSSIONS

STRUCTURAL APPLICATION OF STEEL AND LIGHT-WEIGHT ALLOYS A SYMPOSIUM

Discussion

By Messrs. E. Robert deLuccia, O. J. Horger, A. W. Demmler, Theodore Belzner, J. P. Growdon, Karl Arnstein, A. Christianson, Robert E. Glover, Arthur C. Ruge, Alexander Klemin, Paul E. Gisiger, Russell C. Brinker, Arshag G. Solakian, and H. D. Hussey

E. Robert deLuccia,[145] Esq. (by letter).[145a]—In the popular mind structural aluminum alloy is known only as duralumin, the material from which is fabricated the framework of large rigid airships, such as the *Akron* and the *Macon*; and even in the average engineering mind the thought of using structural aluminum is associated with such special structures as airships and airplanes. In his excellent paper, Mr. Hartmann indicates other uses for this material. He states that but little difference exists in the methods of shop fabrication between aluminum and other structural metals. However, he does state that the workmanship should be of a high quality. It is doubtful whether the average practice for structural steel, say, would be at all suitable for aluminum alloys. The difference does not lie so much in the use of the proper tools but in the care that the shop personnel is willing to exercise in the fabrication of the structure. This does not belong so much in the category of workmanship as it does in the proper appreciation of a new and useful material and in the will to assist the designer to the fullest in his attempt to produce a fine, closely designed structure. Certain lapses, which occur even in the best shops, such as occasional rough handling and nicking of

Note.—This Symposium was presented at the meeting of the Structural Division at Pittsburgh, Pa., October 14–15, 1936, and published in October, 1936, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: December, 1936, by Messrs. E. Mirabelli, R. W. Vose, Raymond H. Hobrock, William F. Clapp, J. C. Hunsaker, Horace C. Knerr, and F. T. Sisco; January, 1937, by Messrs. J. Charles Rathbun and D. M. MacAlpine, Fred L. Plummer, C. F. Goodrich, G. K. Herzog, John H. Meursinge, P. C. Lang, Jr., and W. L. Warner; and February, 1937, by Messrs. Elmer K. Timby, Werner Lehman, Otis E. Hovey, and R. G. Sturm.

[145] Chf. Designing Engr., U. S. Engr. Office, Huntington, W. Va.

[145a] Received by the Secretary January 16, 1937.

material, over-driving and over-heating of rivets, accidental bending of plates and shapes, and consequent straightening, although possibly tolerated without danger to structural steel, would probably damage aluminum material and, therefore, should not be allowed. The rejection of even a small quantity of aluminum material rapidly mounts into a considerable sum of money and, therefore, it is important to the fabricator that he understand clearly the requirements necessary for successful fabrication in this metal. It would appear to be better to stress the proper fabrication of aluminum as such than to compare it to some other material, such as steel. Of course, steel has been used so long as the standard structural metal that it is a temptation to point out the similarity of other metals to its advantages. Nevertheless, it seems important to think in aluminum while working with it, if satisfactory results are to be obtained.

Although the thought of treating aluminum as a sovereign metal is important to the fabricator, it is of the utmost importance to the designer. Mr. Hartmann indicates this somewhat, but it should be emphasized that, due to properties which are peculiarly its own, aluminum should be considered as a quite different material, the various properties of which are subject to successful exploitation in design only when they are thoroughly understood and appreciated. As an illustration, there is the relatively low modulus of elasticity of aluminum which produces deflections that generally would be considered excessive in structural steel. The first thought of the designer, newly introduced to aluminum, is to deepen or thicken the member under consideration in an attempt to reduce the deflection. Such a procedure, of course, subtracts from the property which probably had most to do with the selection of the aluminum, namely, its relatively light weight. The designer, therefore, should understand, and expect fully, that relatively excessive deflection will occur and he should be willing to face this fact squarely, accepting it as proper and usual, and designing his structure accordingly.

A short description of the dam across the Ohio River 9 miles below Gallipolis, Ohio, is given to demonstrate the type of problems that can arise in connection with the design of an up-stream emergency bulkhead for the roller-gates, and their satisfactory solution by the use of a light-weight alloy. The dam consists principally of nine concrete piers (16 ft wide and about 135 ft high) eight gate-hoists, eight steel roller-gates (each with a clear opening of 125.5 ft and a damming height of 29.5 ft) and a concrete sill upon which the roller-gates are seated. The pool above the dam is maintained during increasing flows in the river by raising the gates. In times of flood, the gates are raised completely above the water; as the flow decreases, they are lowered. In the event that a gate is prevented from being lowered due to some accident, the pool is saved by lowering a bulkhead in specially constructed recesses in the piers up stream from the gates. The bulkhead is also used for unwatering the gates for routine inspection, painting, and repairing.

The studies for the design of the dam indicated that the aforementioned size of roller-gates, although unprecedented, would be the most feasible. It

was realized, however, that the relatively long span and high head would require a special solution for the up-stream emergency bulkhead. The problem was further complicated by the fact that the bulkhead would have to be capable of being placed across an opening with the gate raised and passing water through the full 125.5-ft width, under a head of more than 20 ft.

A further consideration in the design of the bulkhead was the method of handling it. It was desired to place it by the use of a whirler derrick-boat. In order that this could be done without endangering the boat, it was necessary that its boom be of sufficient length to enable the boat to lay against an undamaged gate while placing the bulkhead across the opening of the damaged gate. This would result in a boom more than 125 ft long, with the load being lifted at a radius of about 117 ft. It will be readily seen, therefore, that the weight of the bulkhead was a prime consideration.

Various designs for the bulkhead fabricated from structural steel, alloy steel, or structural aluminum alloy were considered. It was found that the structure fabricated from an aluminum alloy would be by far the lightest in

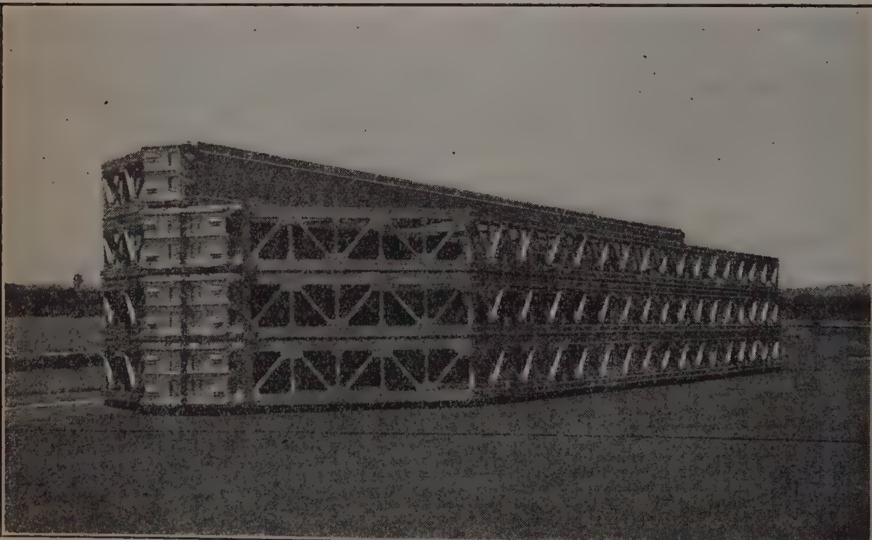

FIG. 51.—SEVEN UNITS, AS TEMPORARILY STORED, OF GALLIPOLIS DAM, OHIO RIVER.

weight and, therefore, this material was selected. To facilitate handling, the bulkhead was separated into seven units of equal size and weight. The weights of the bulkhead, fabricated from the various materials considered, are as follows:

Material	Weight, in Tons: Each unit	Weight, in Tons: Total bulkhead
Structural steel	78	546
Alloy steel	44	308
Structural aluminum alloy	28	196

As constructed the bulkhead has a total damming height of 30.33 ft, each unit having a damming height of 4.33 ft. Fig. 51 (submitted through the

courtesy of the Huntington District, U. S. Engineer Office, Huntington, W. Va.) shows the seven units as stored temporarily. These units are built from two, simple trusses of the Pratt type, which are held apart by channel and angle struts between the chord members. On the inside of the lower chord and extending between the two trusses is a $\frac{3}{8}$-in. skin-plate which serves to make the unit water-tight. Each truss is 127.5 ft long and 12.5 ft deep (measured between working centers). On the outsides of the lower chord of each truss are 4 by 4-in. oak timbers, running the full length of the unit, which effect a seal between the units, and between the lowest unit and the sill, when placed in the dam. Timbers are also provided at the panel points of the top chords to serve as buffer-blocks between the units. On each end of each unit is a housing containing a roller nest which, in turn, contains two rollers $18\frac{1}{4}$ in. in diameter which bear on a steel plate embedded in the down-stream face of the recesses in the piers. The rollers are placed on the down-stream side of the unit and the roller nest is arranged to swivel about trunnion pins so that as the trusses deflect the rollers remain in full contact with the plate in the recess.

Guide rollers are provided on the ends and up-stream side of the housing to prevent binding in the recess in the event that the unit tips in raising or lowering. A specially designed pick-up beam, from which hooks are suspended to engage pins at approximately the third points of each unit, is used to pick up the units.

The truss is fabricated from Aluminum Alloy 27 *S-T* (see Table 7, Item No. 10) with the exception of the bent hip plates and the rivets which are of structural grade steel. The aluminum alloy has the following chemical composition: Aluminum (minimum) 92%; copper, 3.9% to 4.9%; manganese, 0.5% to 1.1%; silicon, 0.5% to 1.1%; tin, 0.03% to 0.07% iron (maximum), 0.9%; magnesium (maximum) 0.03%; and, other elements (maximum), 0.3 per cent. It is used structurally in the heat-treated condition and has the following physical characteristics: Ultimate tensile strength, 58 kips per sq in.; yield strength (permanent set of 0.2% of the initial gage length), 45 kips per sq in.; and, elongation in 2 in. (minimum), 8 per cent.

The following are the maximum stresses, in pounds per square inch, allowed in the design of the Gallipolis bulkhead:

(1) Tension, on net section = 22 000; (2) compression, axial loads, on gross section for $\frac{a\,L}{k}$ less than 67, $= 22\,000 - 219\,\frac{a\,L}{k}$; $\frac{a\,L}{k}$ more than 67, $= 33\,000\,000\left(\frac{k}{a\,L}\right)^2$; and, for truss members, the riveted connections, $a = 0.75$; (3) compression in beam and girder flanges $= 20\,000 - 230\,\frac{L}{b'}$; (4) maximum shear on gross area = 10 000; and, on net area = 13 000, limited always, however, by the formula:

$$s = 12\,000\,000\;t^2\left(\frac{1}{d^2}+\frac{1}{l^2}\right) \quad\ldots\ldots\ldots\ldots(27)$$

in which d = clear depth of web, in inches; and l = clear stiffener spacing, in inches; and (5) bearing = 26 000. In Items (1) to (3) L = unsupported length of flange, in inches; $b' = b\left(c + \frac{t}{d}\right)$; b = width of compression flange, in inches; t = web thickness, in inches; d = over-all depth of beam or girder, in inches; c = 1.0 for symmetrical members, with web and flange rolled or extruded integrally; c = 0.7 for symmetrical members with cover-plate, flanges and web not integral; c = 0.5 for symmetrical members with no cover-plate, flanges and web not integral; and, the value of $\frac{L}{b'}$ was kept less than 40.

The top chord of each truss consists of four 6 by 4 by $\frac{9}{16}$-in. angles, one 16 by $\frac{3}{4}$-in. web-plate, and one $8\frac{3}{4}$ by $\frac{5}{8}$-in. cover-plate arranged in the form of an **I**-section. The cover-plate extends over the four center panels only. The web-plate is cut at the gussets for the diagonals and milled for bearing. The angles are continuous over the gussets. The bottom chord is similar to the top chord, except that the web-plates are not milled at the gussets. Splice-plates are used on the web-plates to transmit the tension in the web across the gussets and are carried a sufficient distance beyond the gusset to take care of the negative moment resulting from the water load on the skin-plate. The diagonals are all rolled or extruded shapes, and range in size from two 8-in. channels weighing 5.78 lb per lin ft to two $3\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{5}{16}$-in. angles. The verticals range from two 10-in. car channels, weighing 9.59 lb per lin ft, to two $3\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{5}{16}$-in. angles. In three cases, the verticals are built-up sections of four 4 by 3 by $\frac{1}{2}$-in. angles, with spreader plates, $6\frac{1}{2}$ by $\frac{3}{4}$ by 8 in. long; four 3 by 3 by $\frac{3}{8}$-in. angles, with spreader plates, $9\frac{1}{2}$ by $\frac{3}{4}$ by 10 in. long; and four 3 by 3 by $\frac{3}{8}$-in. angles, with spreader plates $6\frac{1}{2}$ by $\frac{3}{4}$ by 7 in. long. Drain holes are provided in the webs of both top and bottom chords so that they will drain as the units are lifted clear of the water. The bent hip-plate between the inclined end post and the top chord is of structural steel. This substitution of steel for the aluminum was made in order to avoid bending the latter. A camber of 4 in. which is approximately one-half the total calculated deflection, was provided in each truss. The remaining deflection is cared for by allowing the ends of the truss to rotate freely about trunnion pins on the ends of the nest castings containing the bearing rollers. The trunnion pins are received by bronze-bushed, cast, nickel-steel bearings bolted to the end gusset-plates of the truss with turned steel bolts.

The roller-nest casting is of special steel having an ultimate tensile strength of 100 kips per sq in., a minimum yield point of 60 kips per sq in., and an elongation of 20% in 2 in. After the castings were made they were thoroughly annealed to remove any strains incident to casting. Due to the limited space available for this nest, it was necessary to use relatively high-strength material and, therefore, cast steel was substituted for aluminum. The trunnion pins about which the trusses rotate were cast integral with the roller nest and chromium plated as a precaution against corrosion and to minimize friction.

The bearing rollers $18\frac{1}{4}$ in. in diameter are of cast nickel steel and are bushed in manganese bronze for the nickel-steel pins about which they rotate. Nickel steel was used in lieu of aluminum not only because a stronger material was necessary, but also due to the fact that the relatively small diameter of the rollers and the heavy bearing load made the use of aluminum unsuitable in this instance. The guide-rollers are of cold rolled steel. Their weight is so small relative to the total weight of the unit that it was felt that in this case the more expensive aluminum would not be justified.

Nickel-steel plates are used at the pick-up points due to the higher bearing value of this material. The pick-up pins are also of nickel steel and are adjusted in the units by means of nickel-steel adjusting screws. The seals are of white oak. Wood was chosen for this use due to its easy replacement, shock protection, and capacity to swell under water and so assist in making the seal.

All rivets are of steel and are, in general, $\frac{7}{8}$ in. in diameter. Steel rivets were used because it was felt that aluminum rivets would be impracticable, due to the special heating equipment and close temperature control necessary for their successful use. Where possible all rivets were cold-driven. From examination of both hot and cold-driven steel rivets in aluminum, it appears that less local deformation of the riveted material accrues with the use of the latter method. It is necessary, however, that some provision be made to avoid the use of excessive pressures while cold squeezing. All rivet holes were subpunched $\frac{1}{16}$ in. smaller and reamed $\frac{1}{16}$ in. larger than the nominal size of the rivet.

After the units were fabricated, the aluminum was thoroughly cleaned with a chemical cleaner and rinsed off with clean water. A red iron-oxide primer was then applied. After the primer had dried thoroughly, two coats of aluminum paint were applied. The steel was painted with red lead and two coats of aluminum.

The approximate weights, in kips, of the various materials in one bulkhead unit are as follows:

Structural Aluminum Alloy No. 27 *S-T*	37.9
Structural steel, including rivets	7.3
Structural nickel steel	0.5
Cast nickel steel	3.8
Cast steel (special)	1.9
Miscellaneous bolts, pins, bushings, etc.	2.1
Timber	2.5
Total	56.0

The make-up of the sections of the top and bottom chords and of the diagonals is considerably different from the forms that would have been adopted had steel been used. Large aluminum alloy shapes are not available, ordinarily, due to the lack of rolling equipment. They can be obtained by using steel passes but only at a sacrifice in strength of about 10 per cent. The largest shape that can be extruded is one that can be contained in a

circle 12 in. in diameter, due to the limitations of the size of the ingot. Shapes made by the extrusion process or by rolls made specially for rolling aluminum suffer no reductions in strength. Therefore, only shapes that could be contained in a circle 12 in. or less in diameter were used. The relatively high base cost of this material made it economical to allow considerably more fabrication per pound than is ordinarily the case for steel. For this reason, even small sections were built up instead of using a single shape of somewhat greater weight. If fabricated from structural steel, the chords would probably be made up of two channels, a fill-plate, and a cover-plate. The design adopted, however, while increasing fabrication utilized the minimum possible weight of material.

The deflection of the unit is calculated to be approximately 8 in., which is about three times the deflection which would be expected for steel. However, by cambering the trusses 4 in. and by using a swiveled roller nest in the housings at the supports, it was possible to accept the apparently high deflection. As might be expected, secondary stresses of considerable magnitude occur in the trusses when they are under load, and consideration was given them in the design.

O. J. HORGER,[146] ESQ. (by letter).[146a]—Those responsible for the non-failure of materials in their application to structural and machine parts find greater need to-day for an intelligent use of stress theories. For this reason Mr. Karpov's paper, reviewing the knowledge available, is of considerable interest.

It is not clear from observation of Fig. 3(*a*) for the un-notched tension specimen why the stress is not uniformly distributed for values within the yield point[147]. The same question applies to Fig. 9 for the case of a rectangular bar subjected to bending where the bending stress within the yield point should be linear.

The fatigue diagrams presented in Fig. 10 are apparently based on plain specimens; that is, without stress concentration. It would be interesting to see comparable diagrams on the basis of notched specimens. Such diagrams would be the ones actually required in most design problems since failure usually occurs in members due to local stress concentration, such as fillets, holes, etc.

Fig. 10(*b*) indicates that the lighter metal has a strength about equal to nickel steel on a weight-strength basis. When the lighter alloy is used the section dimensions must be increased over the steel so that a size effect is also involved in the calculation of comparable fatigue strength. This factor of size effect may be small, considering plain specimens, but with stress concentration it is important and must be considered. The writer is familiar with size-effect factors using steels as given in published literature, and would like to ask whether the author can give similar data on the light alloys?

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[146] Research Engr., The Timken Roller Bearing Co., Canton, Ohio.

[146a] Received by the Secretary January 14, 1937.

[147] "Photo-Elasticity", by Coker and Filon, 1931, Cambridge Univ. Press, England; also, "Theory of Elasticity", by S. Timoshenko, 1934, McGraw-Hill Book Co., New York, N. Y.

Considering the effect of the aforementioned factors of stress concentration and size effect does the author believe that these considerations would alter the relative comparison shown in the diagrams of Fig. 10(*a*) and Fig. 10(*b*)?

A. W. DEMMLER,[148] ESQ. (by letter).[148a]—Attention has been called by Mr. Beard to U. S. Navy Specification No. 48 *S* 5*e* covering manganese-vanadium steel in connection with steels suitable for fusion welding. The 0.18% carbon, 1.45% manganese, and 0.25% silicon, listed in this specification, are maximum values[148b]. The presence of vanadium in this combination results in distinct grain refinement with uniformity and also serves to insure toughness and freedom from sensitive hardenability.

Dr. Bain has also mentioned these features and, in discussing orally the cumulative effect of various alloy additions, made the true and interesting point that simple arithmetic does not give the full picture in that "two plus two plus two may equal seven" in the behavior of the final product. In the absence of vanadium, 1.45% manganese would be frowned upon for fusion welding; yet with 0.10% to 0.12% vanadium a piece of 0.5-in. plate water-quenched from 2 400° F, and machined free of any decarburization, will have a Brinell hardness number of only 400, while air-hardening from this temperature will give a Brinell hardness of about 250. Therefore, any danger of encountering a glass-hard condition from a welding operation is out of the question.

THEODORE BELZNER,[149] AFFILIATE, AM. SOC. C. E. (by letter).[149a]—So carefully has Mr. Templin covered the methods of operation, and the precautions, required in the manipulation of the various types of strain-gages for precise work on engineering structures and their models, that little can be added. Possibly a few references could be made to the measurements required after the preliminary work.

The skill required to obtain reliable results in strain-gage measurements is not in the possession of all; and the writer agrees with Mr. Templin, that,

> "In the testing of structures, the errors arising from the personal equations involved in the manipulation of the instruments used, must be considered, in addition to those already mentioned. The magnitude of these errors becomes less, at a diminishing rate, as the experience of the observer increases."

During 1913 and 1914, a comprehensive series of extensometer investigations was made in connection with the strengthening of the end spans of the Williamsburg Bridge, by the Department of Bridges (later the Department of Plant and Structures), City of New York, in co-operation with

148 Metallurgical Engr., Vanadium Corp. of America, Bridgeville, Pa.

148a Received by the Secretary January 15, 1937.

148b Correction for *Transactions:* In Table 15 (*n*), Column (9), Item No. 52, of the Symposium, insert decimal point, thus, "0.08/0.18."

149 Insp. of Steel, and Bridge Insp.-in-Charge, Brooklyn Bridge, Dept. of Plant and Structures, City of New York, Brooklyn, N. Y.

149a Received by the Secretary January 19, 1937.

the National Bureau of Standards, under the auspices of the late James E. Howard, Engineer Physicist, on the important truss members at the main towers and legs of the Brooklyn intermediate towers.

During these investigations, extending over a long period of measurements, a Howard extensometer of 20-in capacity was used throughout the entire test; and the precision attained with the strain-gage proved to be an invaluable aid, and its merits were illustrated in certain vital operations, especially with the various stages of wedging, and in the transferring of stress from the old to the new diagonal members of the end trusses.

The results obtained firmly convinced the writer that the most important objective to be considered in taking stress-strain measurements (other factors being equal) is to obtain consistent results; and in order to secure such results (which are absolutely essential), persistent patience and painstaking thoroughness, combined with good judgment on the part of the observer, are required, until he experiences little or no difficulty, and it is upon the observer that the value of such measurements will depend.

J. P. GROWDON,[150] M. AM. SOC. C. E. (by letter).[150a]—The engineer who is concerned with the design of structures, such as bridges, rather than the design of structural parts of machines, such as dragline booms, will be interested in the use of strong aluminum alloys when these alloys will: (1) Permit him to design and build a structure that would not otherwise be possible; and (2), when it will permit him to design and build a more efficient structure than can be obtained by the exclusive use of other material.

The first situation occurs in certain fields, such as that of aircraft design, but will be rare in the structural field because all the structures which are now required can be successfully constructed of other materials. The second situation will be more frequently encountered by the structural engineer, who can, in most cases, measure the efficiency of his design in dollars, either in the form of reduced maintenance cost or of less capital investment.

Reduced maintenance cost can often be achieved by utilizing the high resistance to corrosion possessed by aluminum alloys of the structural group as compared to alloys of other materials. Less capital investment can be achieved in many specific cases by utilizing the bulk-weight ratio as well as the high strength-weight ratio of these alloys.

The extent to which aluminum alloys can be used economically in any given structure will depend upon their cost, relative to the cost of other materials which can be used for the same purpose, the type and size of the structure, the physical characteristics of the alloy, the skill with which the designer makes use of it, and the price at which it can be purchased.

To-day, the cost of aluminum per pound used to replace low alloy steels will be five or six times the cost of such alloy steels. This cost differential effectively limits the use of structural aluminum to those places where 1 lb of the newer material will save several pounds of steel.

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[150] Asst. Chf. Hydr. Engr., Aluminum Co. of America, Pittsburgh, Pa.

[150a] Received by the Secretary January 21, 1937.

This use of structural aluminum has been well illustrated in bridges, both new and reconstructed. Referring to new bridges, of the suspension, cantilever, or truss type, this limitation will mean that aluminum can be used only in those parts of the structure where the saving in dead weight will be most effective in reducing the weight and cost of other parts. In new short-span bridges such a saving is small, whereas in new long-span bridges the possible saving in dead weight (and, hence, in cost) becomes very large; so that one may say that in short-span bridges no aluminum can be used economically for structural parts, but as the length of span increases, the possibility of saving money by its use will also increase. Furthermore, the longer the span, the greater will be the number of places in which it can be used to advantage.

Various types of bridges will require aluminum in different parts of the structure in order to secure the most economical result. In a suspension bridge, it will be used first in the deck and with longer spans in the stringers, floor-beams, and stiffening trusses. In a cantilever bridge aluminum would find its most profitable use in the suspended span, where a saving in weight would reduce progressively the weight of the cantilever arms, the anchor arms, the towers, and the size of the foundations. Thus far, aluminum has found little utility in truss bridges except for railing and decorative parts.

In a bascule bridge a pound of weight in the outer end of the leaf requires several pounds of material in other parts of the structure to support it. A lift span, constructed entirely, or in part, of aluminum, would require a lighter counter-weight, lighter towers, smaller motors, and less power to operate it. In all bridges of the bascule or lift type aluminum deserves careful consideration from the standpoint of economy and efficiency.

In the rehabilitation and reconstruction of old bridges the high strength weight-ratio of aluminum has found a most effective use. Many old bridges, inadequate for modern traffic, can thus be reconstructed so as to reduce the dead load, increase the traffic capacity, or improve the character of the roadway. In some cases all three results may be accomplished.

From the many aluminum alloys available Mr. Hartmann has selected a single alloy of the duralumin type, (17*S*-*T*), and has largely limited his discussion to that single alloy. In thus selecting Alloy 17*S*-*T* as the typical alloy he has been wise since this alloy was the first of its type to be developed and has long been established in the structural field. As pointed out by Mr. Hartmann, there are many other alloys available to the structural engineer, and it is believed that certain of the newer alloys now available have physical characteristics which particularly adapt them for use in such structures as bridges. Among these characteristics may be mentioned: Alloys 53*S*-*T* and 27*S*-*T*, included in the paper by Messrs. Jeffries, Nagel, and Wood (see Table 7, Items Nos. 10 and 13). They differ from Alloy 17*S*-*T*, because of the lower tensile and yield strengths of the latter, (which are, respectively, 33 and 20 kips per sq in.), but more especially because of its extraordinary resistance to corrosion.

Item No 10, Table 7, differs from Item No. 6 in that it has a greater yield strength (50 kips per sq in.) and a somewhat smaller percentage of elongation (9 to 13%). These characteristics make Item No. 13 particularly suitable for applications where maximum resistance to corrosion is desirable, but where maximum strength is not required. Bridge railing is an example of such an application. The characteristics of Item No. 6 make it particularly suitable for those applications in which the maximum strength-weight ratio is desirable.

To secure satisfactory results, it is not sufficient merely to substitute an aluminum member for a similar steel member. The designer must take advantage of the particular properties of the material so as to utilize these properties to the best advantage and to minimize the effect of disadvantages which are inherent in certain of them. In addition to the ultimate strength and the yield strength, the designer must keep constantly in mind the modulus of elasticity, the temperature coefficient, the weight, and the cost. The relatively high price of aluminum makes it necessary for the designer to have accurate knowledge of the properties of the material, the loads to which the structure will be subjected, and the stress which these loads will produce in every part of the structure. He must exercise his skill, ingenuity, and best judgment, in addition to his knowledge, not only to see that every part of the structure is adequate for the work which it must do, but that no material is wasted.

A properly designed structure utilizing aluminum in a greater or less degree will be different from a steel structure serving the same purpose, because the properties of the material are different. As an illustration of how the properties affect the design, it is interesting to note the effect of modulus of elasticity, which, of course, in an aluminum structure having a modulus of elasticity of 10 000 000 lb per sq in. will produce deflections much greater than in an identical steel structure having a modulus of elasticity of 29 000 000 lb per sq in. In many cases, this excessive deflection can be reduced within satisfactory limits by utilizing continuity wherever possible in the design. Other factors in which the design of an aluminum structure will differ from that of a steel structure will occur to the engineer who thinks seriously on the subject.

The Smithfield Street Bridge, across the Monongahela River, at Pittsburgh, Pa., was reconstructed in 1933 by replacing the entire wood and steel floor system with an aluminum floor system of Alloy 27*S*-*T*.[151] It has functioned satisfactorily since reconstruction and is an excellent example of the successful application of one of the newer light alloys in the bridge field.

KARL ARNSTEIN,[152] ESQ. (by letter).[152a]—An interesting presentation of the most important phases of models and model testing is contained in the paper by Mr. Templin. The modern trend toward light-weight designs of larger structures, requires that more consideration be given to the subject of general

[151] *Civil Engineering*, March, 1934.

[152] Chf. Engr., Aeronautical Dept., The Goodyear Tire & Rubber Co., Akron, Ohio.

[152a] Received by the Secretary January 28, 1937.

buckling. Mr. Templin refers to this problem in a general way only. It should be emphasized that, in order to obtain reliable general information on buckling in scaled down or scaled up models, very rigid specifications are required in designing the model and model members. Not only can general buckling be sensitive to the axial, bending, torsional, and shear characteristics of its individual members, but it is essential that the unit linear deformation (and, hence, also the relative angular deformation), under any system of forces or moments, be the same as the corresponding unit linear, and relative angular, deformation in the corresponding prototype member, under the properly scaled corresponding forces and moments.

For most types of engineering structures, the shear stiffness of its individual members is sufficiently large to have only a small effect on the general stability (unless a built-up section is represented by a single member in the model) and can be neglected. To meet the remaining requirements, it is necessary, for reasons mentioned by Mr. Templin, that the model member be designed so that the axial, bending, and torsional stiffness can be varied independently. Such a model girder has been developed and used by the writer's company and has been explained, together with other interesting comments on this subject, by Mr. L. H. Donnell[153]. The application of this type of model representation may remove some of the objections raised by Mr. Templin in his valuable paper.

A. Christianson,[154] Esq. (by letter).[154a]—The weight savings accomplished through the use of structural aluminum in the framing of car bodies have proved to be a substantial addition to the weight saved through the other major application of aluminum in this field, namely, inside finish and fittings. The extra cost of providing these weight savings is offset by the savings in power and improvements in performance. The trend away from the conventional steam locomotive, of course, has accelerated the demand for light-weight construction.

Aluminum alloys are well adapted to light-weight car construction because they are available in forms that fit the needs of the car builder. Large-sized sheets with a high degree of flatness are readily obtained, and the shapes, particularly the special extrusions, meet all the requirements of form, thickness, and length.

The low unit weight of the aluminum alloys permits light-weight construction without drastic reductions of thickness of sheet and shapes. This has been found to simplify both design and construction. The car designer, having available a greater volume of metal, is able to distribute it effectively in the members and still not depart radically from conventional sizes which have been established through years of experience in car-building practice. The use of the greater volume of light-weight metal and the resulting thicker sections greatly reduces the tendency for buckling failures and other compli-

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[153] "Model Measurements and Airship Stress Analysis", pub. in "Report on Airships Forum", July 25-26, 1935.

[154] Chf. Engr., Eng. Dept., Pullman-Standard Car Mfg. Co., Chicago, Ill.

[154a] Received by the Secretary January 28, 1937.

cations due to instability. The pleasing and substantial external appearance which always accompanies freedom from wrinkles is readily obtained in aluminum construction with no extra effort in the shop.

Mr. Hartmann makes some interesting comments on workmanship, indicating that aluminum construction should receive careful attention in the shop in order to help approach the degree of perfection for which the designer is striving. The importance of this cannot be too highly emphasized. It has long been the writer's contention that no construction can function as the designer intended, unless the shop work is perfect. Excellence in fabrication is of importance to aluminum construction for the same reasons that it is important to all other construction, and is accomplished in exactly the same manner. Aluminum alloy construction presents almost no new problems to the shop experienced in producing a high quality of workmanship in other metals.

ROBERT E. GLOVER,[155] ESQ. (by letter).[155a]—In his discourse on "The Extent of Present Theoretical Knowledge" Mr. Karpov comments on the lack of information regarding the electric and magnetic forces of the electrons, and expresses the belief that the fundamental avenue of approach to the problems of stress is now closed because of a lack of essential basic information. He then refers to the "theory of stress" and concludes that, because no general solution of the differential equations is known, the designer is forced to fall back upon a number of more or less reliable assumptions among which the most important are that the material follows Hooke's law and that the stress follows a straight line distribution. The writer can not agree that the designer need be reduced to such straits. It is well known among students of the subject that the general solution of a partial differential equation may be of little use in cases where a particular problem must be solved. Since the differential equations of elasticity are of this type the lack of such a solution need not be cause for great concern. The question of the validity of Hooke's law does not admit of a general answer applying to all materials, but with regard to a given material, may be answered at the nearest testing machine.

To the writer's knowledge, there are at least four good, sound, carefully written texts on the theory of elasticity which contain a large number of solutions of the elastic equations, many of them illustrated with solved examples for particular cases. With these at hand, supplemented, perhaps, by some other excellent available texts on special subjects, such as photo-elasticity, plasticity, slab theory, elastic stability, and the contributions which are constantly appearing in technical literature, the designer should not find his problem of stress determination hopeless.

The uniqueness theorem[156] has a bearing on some of the statements made by Mr. Karpov. This theorem was published in 1859 by the distinguished

[155] Engr., U. S. Bureau of Reclamation, Denver, Colo.

[155a] Received by the Secretary February 2, 1937.

[156] "Mathematical Theory of Elasticity", by A. E. H. Love, Fourth Edition, Paragraph 118.

investigator, G. Kirchhoff, whose name is so well known in the electrical world. He proved that under certain stipulated conditions (which, indeed, apply to the great majority of engineering structures), the differential equations of elasticity can have no more than one solution. If the existence theorem were also available, the case would be complete, and it could be inferred that the elastic equations, as they exist to-day, solved in conjunction with the supplementary conditions imposed by any particular problem are completely adequate under the postulated conditions. There seems to be small reason to doubt that this is actually the case, but if the required solution is known or can be found the existence theorem may be dispensed with, since the possession of a solution is ample proof of existence, and the designer can proceed with the assurance that, in his case, the problem is properly set. Furthermore, since no other solution can exist, he also has the assurance that the results he arrives at can not be altered by whatever information some later investigator may succeed in prying out of the electrons.

It is true that the theory of elasticity is mathematically difficult, but it is also true that so many of these difficilties have been overcome by able investigators that the charge of practical uselessness can no longer be sustained. The writer has had occasion to apply the methods of the theory of elasticity to a number of problems and has obtained very satisfactory results. The Kirchhoff theorem has been of great assistance in connection with the trial-load method for the design of arch dams, by making it possible to determine what constitutes an adequate analysis.[157] These methods were also applied, under the writer's direction, to the problem of developing formulas for designing the supporting rings for the penstocks at the Boulder Dam, which is located about 30 miles southeast of Las Vegas, Nev. These formulas were obtained and used with entire satisfaction, and the results have been confirmed by extensive tests both in the laboratory and on the penstocks themselves in the field. These and similar formulas have been used more recently in the design of the Malheur siphon on the Owyhee project in Eastern Oregon. This pipe is 80 in, in diameter and is constructed with 60-ft spans, without intermediate stiffener rings.

ARTHUR C. RUGE,[158] ASSOC. M. AM. SOC. C. E.(by letter).[158a]—A remarkably complete and lucid discussion of the types and purposes of tests of engineering structures and their models has been presented by Mr. Templin. Although he does not specifically make a critical comparison of the testing of full-sized structures with that of their models, one feels that he is doubtless aware that defects in the model method are sometimes over-emphasized in relation to those defects which are inherent in the testing of many full-sized structures. The principal difference in this respect lies in the relative ease with which the model test conditions can be controlled and varied.

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[157] "Fundamentals of the Trial Load Method for the Design of Arch Dams", by R. E. Glover, presented as a thesis to the Univ. of Nebraska in partial fulfillment of the requirements for the degree of Civil Engineer.

[158] Research Associate in Seismology, Dept., Civ. and San. Eng., Mass. Inst. Tech., Cambridge, Mass.

[158a] Received by the Secretary February 4, 1937.

As a case in point, one might consider the dynamic tests of structures, which are becoming important as a means of interpreting the effects of earthquake motions. Consider a tall masonry chimney in which it is desired to study the vibration periods for the purpose of checking theoretical calculations. If the model method is selected, difficulties arise only in securing the proper material to represent the masonry and the foundation material. These difficulties may be very serious; but they are not insuperable.

If, on the other hand, one chose to test the actual chimney in the field, one might be misled at first by the apparent simplicity of the problem, but sooner or later one would discover defects more serious than in the model technique. The first great difficulty would be in generating the modes of vibration, and, assuming that has been done, one cannot know if the periods measured will be maintained for the large amplitudes which are really the most important, but which the investigator dares not, or cannot, produce in the test. For example, a chimney laid up with completely bare bricks would exhibit a set of natural periods not greatly different from those of the same chimney laid up in good mortar; but the dynamic behavior under large amplitudes would be vastly different.

Next, it is impossible from field test data to separate the effect of the foundation from that of the masonry in producing the observed periods, unless broad assumptions are adopted as to the correctness of the very theory to be checked. In such a problem the model method is not nearly at such a disadvantage as it might at first seem. In the model, the conditions of amplitude, stress, and foundations can be controlled and varied in such a manner as to give a positive check on the theory involved. To a less extent the same arguments will be found to hold for certain purely statical tests of structures.

It is not to be ignored, of course, that a tremendous amount of work remains to be done in the development of model materials and artifices for overcoming difficulties in similarity; but it needs to be emphasized that there may be (and often are) pitfalls in the testing of actual structures which are likely to be overlooked entirely. In this connection, Mr. Templin very rightly points out the importance, at times, of considering the model itself as a structure, without assuming a simple relation to its prototype.

The statement (under the heading, "Test Methods: Models Made of the Same Material as the Prototype") that a "photographic" model of a gusseted truss would not be truly similar, needs to be clarified somewhat. Although it is quite true as stated that a geometrically scaled model will not result in the same scale ratio for areas as for moments of inertia, this fact alone cannot preclude the realization of perfect similarity conditions. In fact, unless these ratios are different, true similarity cannot exist in the usual sense of the word.

If, for simplicity, the assumption is made that the connections are welded, or that the riveted connections do not slip, it is difficult to understand why Mr. Templin makes the statement that "the degree of end restraint will not be the same as in the prototype, and the behavior of the various members of the model, under combined axial loads and bending, will be different from that of the corresponding members of the prototype." The elastic action of a

gusset-plate cannot be said to be of a different nature from that of a beam or strut, although the stress distribution may not be equally simple; and, fortunately, models do not have the means to avoid complex stress distributions as is commonly done in analytical work.

Mr. Templin's point is very good in regard to the limitations of models; but he has chosen an unfortunate example for illustration. To present a better picture of the difficulties of scale effects in similarity problems, one needs only to consider structures in which more than one type of physical forces act simultaneously. In the gusseted truss only elastic forces are involved. In the case of a structure like an elevated water tank (considered dynamically) both gravity and elastic forces are found interacting in such a way that a "photographic" model actually fails to represent the prototype to a marked degree.

The classical dimensional analysis in the water-tank problem promptly leads to the result, $\lambda = 1$; or, the model must be made full size to give true similarity. By proper distortion of scale, however, one easily preserves the similarity in spite of the first result, it being only necessary to find the means of keeping the same force scale for both gravity and elastic forces. To be sure, in practical work of this kind, one may have to be content with securing true similarity only in those features of the behavior he desires to study, but there can scarcely be objection to that.

It is hoped that, having laid such an excellent foundation, Mr. Templin will find it possible, some time, to extend his present paper to a study of the comparative merits of full-sized and model tests by analyzing certain specific examples from the standpoints of both.

Alexander Klemin,[159] Esq. (by letter).[159a]—In the construction of aircraft one encounters both complicated structures and complicated distributions of loads. The loads applied are air pressures and not only is the distribution peculiar but it varies with each condition of flight. The complete structure is very frequently tested to destruction.

Models are used infrequently. In the structural testing of aircraft, the technique is not too well defined and methods of determining stresses and strains are likely to be improvised for a particular construction. Those engaged in aircraft engineering have learned much from the Civil Engineering Profession and Mr. Templin's paper will add to such lessons. Every aeronautical engineer who makes structural tests should follow the principles enunciated.

Paul E. Gisiger,[160] Assoc. M. Am. Soc. C. E. (by letter).[160a]—In connection with the very informative paper on stainless high alloy structural steels by Mr. Morris, it may be of interest to cite a recent application of non-corrosive steel in the field of hydraulic engineering.

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[159] Prof., Daniel Guggenheim School of Aeronautics, Coll. of Eng., New York Univ., New York, N. Y.

[159a] Received by the Secretary February 4, 1937.

[160] With Pennsylvania Water & Power Co., Baltimore, Md.

[160a] Received by the Secretary, February 4, 1937.

In the hydro-electric power plant of the Pennsylvania Water and Power Company at Holtwood, Pa., the trash screens which protect the turbine intakes against floating logs, trash, etc., and which were installed between 1910 and 1913, are now (1937) approaching the end of their useful life. Screening equipment consists of cast-iron guides bolted to the piers between individual intake openings (of which there are four for each of the ten main turbines and one for each of the two exciter turbines) and of individual screen units (five per opening or twenty per turbine), which slide in the guides and can be removed, changed, or replaced easily with overhead cranes.

The original screen units are made of structural steel channel frames, 11 ft high and 9 ft 6 in. wide. The screen bars are $4\frac{1}{2}$ by $\frac{3}{8}$ in. flat bars spaced $4\frac{5}{16}$ in. and held to the frames with bent and slotted attachment plates and bolts. Each such unit weighs about 4 500 lb so that the total of 210 units represents a considerable tonnage.

The established maintenance practice is to sand-blast and repaint the screens once every four years. During recent years it became increasingly necessary, however, to add miscellaneous repairs to this maintenance schedule and, by 1935, it was evident that replacement of at least a part of the screens could no longer be postponed.

An investigation was then made into the use of materials which would require less maintenance and possibly would last longer than the structural steel. In addition to several kinds of corrosion-resisting alloy steels this investigation included bronze, copper, and aluminum. In order to be economically justified a material other than low carbon structural steel had to give reasonable assurance that the increased first cost would be offset by less maintenance and a longer life.

Comparative estimates in which such factors are of large influence are naturally somewhat uncertain and will have to be checked by experience. They were conclusive enough in this instance to decide in favor of new screens with a frame of structural steel as before, but with screen bars made of high-strength chromium alloy of approximately similar composition as Item No. 2, Table 4. Wearing strips of the same material were also provided along the contact line between screens and guides. This resulted in somewhat more than one-half the entire surface area of the screens being non-corrosive and the parts which still have to be painted are those which suffer the least wear are easiest to paint.

Each of the new screens which have the same outside dimensions as the old ones includes 1 050 lb of corrosion-resistant alloy steel and 2 350 lb of standard structural steel. The total weight of the new screens, therefore, is only 75% of the old ones which fact in itself is very desirable because it makes for easier handling. The saving results mostly from the higher strength of the alloy material, but also from the all-welded construction.

Another advantage resulting from the use of high-strength material as well as from welding is the very considerable reduction in the area that obstructs the water passage. This is quite evident when pictures of the old and new screens are compared, and the improved hydraulic effect is

expected to be further emphasized by decreased friction between water and screen bars which do not become roughened by corrosion.

The welding of the high-strength chromium steel and especially its connection by welding to a different material were made the subject of close investigation. A number of types of welding rod were tried and welds examined and tested. The results indicated that generally a somewhat better penetration is obtained if the welding rod is of a composition similar to the base metal and that, therefore, the joints between ordinary steel and alloy steel for which a coated alloy steel electrode was used, had to be welded with more than the usual care. During all these investigations and tests the manufacturers of non-corrosive steel were most helpful in co-operating to arrive at the most suitable materials and methods. Of the old screens in the Holtwood Plant, 25% have now (1937) been replaced and it is intended to replace the remainder progressively during the next few years.

There is no doubt that hydro-electric structures offer a wide field for the application of non-corrosive high strength steels and that their use for such structures will increase rapidly as knowledge concerning these metals advances and as production on a larger scale will make it possible to lower their cost.

RUSSELL C. BRINKER,[161] JUN AM. SOC. C. E. (by letter)[161a]—Engineers are given a clear-cut analysis of the possibilities and limitations of light-weight alloys in the papers of this Symposium. Several statements from the various papers might conveniently be linked for the purpose of greater emphasis.

In their "Summary," Messrs. Bain and Llewellyn state that, "* * * corrosive attack does not reduce the effective metal section in proportion to the thickness, but irrespective of section." Simple arithmetic thus shows that with the thinner sections of material with higher unit strength, the relative effect of corrosion is greater since a $\frac{1}{16}$-in. reduction in thickness due to rusting of a $\frac{3}{8}$-in. plate means a loss of 16.7 per cent. The same loss from a similar $\frac{1}{2}$-in. ordinary steel plate is equal to a section loss of only 12.5 per cent. With total strengths in direct proportion to the thicknesses, the high-strength steel would thus suffer more. If, however, the thickness was governed by other factors, such as column action, stiffness, etc., the effect might be more, or, in exceptional cases, less serious than indicated by these values.

Mr. Aston illustrates substantially the same condition (see heading, "Low-Alloy Structural Steel") and suggests a design solution in his analysis by means of a "unit corrosion-merit" value, his statement being, "in effect, therefore, in the selection of structural materials, the benefit to be derived, as far as it relates to corrosion, will be only to the extent that the corrosion-merit ratio exceeds the strength ratio for the metals under consideration."

Incidentally, Mr. Karpov's statement (see "Application to Actual Designs: Safety Factors and Working Stress") that, "in all engineering structures the factor of safety decreases as time goes on", will apply to the case of structures weakened by corrosion. Part of the spread between the design

[161] Exchange Teacher in Civ. Eng., Worcester Polytechnic Inst., Worcester, Mass.

[161a] Received by the Secretary February 5, 1937.

stress and the elastic limit is assumed to take care of this feature along with secondary stress, increased loading, etc. Hence, in choosing a design stress, the corrosive resistance of a new alloy steel should be known. In the past only too often paint maintenance has been assumed by the engineer, but failed to come up to expectations. Rusted stringer flanges and connection angles, corroded to a point that makes support of only the dead load appear miraculous, are not uncommon sights. A classic example is that of a steel plate girder bridge that carried traffic practically to the time of its replacement when the deteriorated web could be punched through by a single blow of an ordinary sledge hammer. Truly, corrosive resistance will have to be specified along with ultimate strengths as alloys of greater and greater strength are put on the market.

The esthetic possibilities suggested by the alloy steels are not to be overlooked. Many American structures appear less graceful than similar European designs, at least in past constructions. The attitude in the United States has inclined toward the liberal use of materials and the limited use of the designers' time, whereas European economy has dictated the need of ample design time to analyze and produce a light and materials-saving structure. Perhaps with the double incentives of lighter but costlier material, the clumsy appearance of many "railroad-type" bridges will gradually disappear from future highways. A deterring factor, however, is that, in general, the longer structures which have received the necessary close design scrutiny have attained greater esthetic beauty than the intermediate or short spans; but it is in the former and not in the latter classes that the high-strength steels will find their greatest usefulness. There has been no excuse for bulky structures in the long-span field where an extra pound of dead load weight at the bridge center would require as much as 5 lb of added metal to carry it. Certainly, if the use of alloy steels should extend to the shorter spans, many of the wasteful "standard designs" would be eliminated by the more costly material.

ARSHAG G. SOLAKIAN,[162] ESQ. (by letter).[162a]—Dealing with several phases of photo-elasticity (such as history, application, limitations, methods, apparatus, model materials, etc) the valuable paper of Mr. Brahtz is most timely. It reflects the wide interest shown in the last decade, in America as well as abroad, in this interesting optical method of measuring stress intensities in transparent models. The writer's discussion is intended simply to supplement certain parts of Mr. Brahtz's paper, and he has preserved the order of representation used by the author.

It is true that Sir David Brewster has been universally recognized as the founder of photo-elasticity, because of his discovery in 1816 of the stress-optical phenomenon known as double refraction of polarized light in transparent isotropic materials under stress. It was Erasmus Bartholinus, however, who, in 1669, first observed the phenomenon of polarization by the

[162] Lecturer in Civ. Eng., Columbia Univ., New York, N. Y.

[162a] Received by the Secretary February 12, 1937.

double refraction of ordinary light in its passage through a uni-axial crystal, such as Iceland spar, a material used extensively for polarizing prisms in polariscopes. It is evident that without polarized light, Brewster would have not found it possible to observe the double-refraction effect in a stressed plate.

In America, the pioneers in the art of photo-elasticity have been Carus A. C. Wilson[163] and the late Charles Lee Crandall, M. Am. Soc. C. E., and Anson Marston,[164] Past-President, Am. Soc. C. E., but it has been due to the numerous valuable researches and publications of Mesnager, in France, and Filon and Coker, in England, that photo-elasticity has become a practical and useful method for the analysis of stress distributions in machines and structures of a two-dimensional nature.

Mr. Brahtz refers to a paper[165] by the late A. H. Beyer, M. Am. Soc. C. E., and the writer, in connection with the possibility of stress analysis in reinforced concrete through composite models, made of bakelite and cast with metal rods of circular cross-section embedded. As a necessary improvement on this type of test, the circular-section reinforcement may be replaced advantageously by flat strips, having a width equal to the thickness of the model. This modification will distribute the metal uniformly across the entire thickness and also will eliminate any three-dimensional stress effect on particles in the immediate vicinity of such rods as were formerly proposed. It may be of interest to add, in this connection, that a photo-elastic material of preferably liquid form (such as marblette,[166] for example) that can be handled easily under laboratory conditions as a matrix for the models, and also affords good adhesion with the metal inserts and develops the least amount of initial strain effects after being cast, will be of great help to investigators in work of this nature.

Mr. Brahtz emphasizes the necessity for developing a cement that can be used to combine models of the same or different elastic properties, thus opening a wide field of research in a great many problems of practical importance to engineers, such as shrinkage stresses in dams, welded connections, built-up structural sections, space frames with rigid joints, etc. It should be noted that such a cement, made of liquid marblette and hydrochloric acid and setting under room-temperature conditions, has been used by the writer in the qualitative analysis of shrinkage stresses in dams, as reported in a discussion[167] of a paper by Howard G. Smits, Jun. Am. Soc. C. E.

A special type of polariscope (with aluminum-coated mirror reflectors instead of lenses) has been advocated for instruments producing a large field of parallel rays, with the minimum space and lowest cost as governing factors. The writer doubts that such instruments with mirror reflectors will command much popularity, especially in view of the recent development of

[163] "The Influence of Surface Loading on the Flexure of Beams", *Philosophical Magazine*, December, 1891.

[164] "Friction Rollers", *Transactions*. Am. Soc. C. E., Vol. XXXII (1894), p. 99.

[165] "Photo-Elastic Analysis of Stresses in Composite Materials", *Transactions*. Am. Soc. C. E., Vol. 99 (1934), p. 1196.

[166] "A New Photo-Elastic Material,,' by A. G. Solakian, *Mechanical Engineering*, December, 1935.

[167] *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 937.

large-sized and moderately priced artificial polarizing media in the United States. A polariscope having a field of 10-in. parallel rays, for example, has been designed recently by the writer, using polarizing glass plates both for the polarizer and analyzer, with a total length (not including the screen) of about 6 ft. By the addition of a glass plate (at 45° with the axis of the instrument) and a reflection mirror, with the second half of the instrument turned 90° (as shown by the dotted-line arrangement in Fig. 52), this

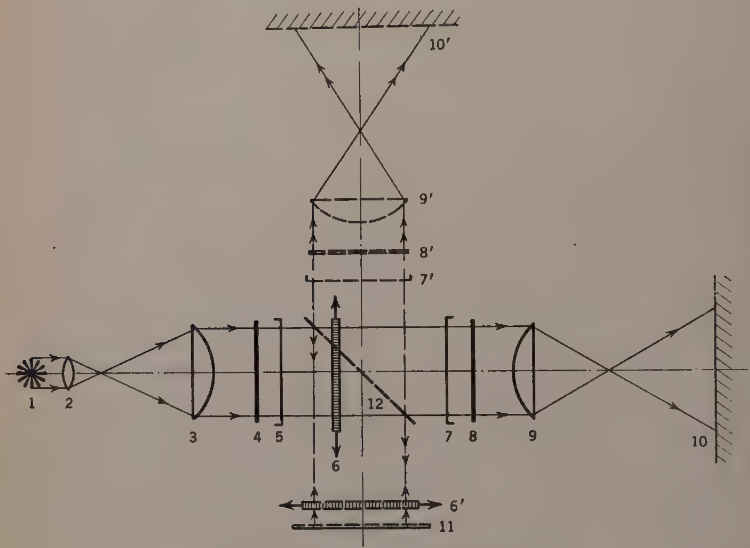

FIG. 52.—UNIVERSAL POLARISCOPE.

"transmission" type of polariscope is easily transformed into the "reflection" type, which latter is of extreme importance in a great many specific cases that could not be handled by the transmission type of polariscope.

Referring to Fig. 52, when the apparatus is set up as a transmission type of polariscope, the light source is at Point 1; Point 2 indicates the condenser lens; Points 3 and 9 the lenses for parallel light; Point 4 is the polarizer; Points 5 and 7 are quarter-wave plates; Point 6 is the model; Point 8 is the analyzer; and Point 10, the screen. In the converted set-up (the reflection type), Points 1 to 5, inclusive, are the same as before; Point 12 is a glass plate; Point 6′ is the model; Point 11, the mirror, and Points 7′ to 10,′ inclusive, are the same as in the transmission type, except that they are turned through 90 degrees.

The idea of evaluating the sum of the principal stresses at a point, by measuring the lateral strain of the stressed model at that point was first suggested by A. Mesnager[168] who designed and used a lateral strain-gage similar to that used by Vose. Coker's lateral extensometer is a more practical type of Mesnager's instrument. The optical flat-model combination for

[168] "Sur une Methode pour déterminer a l'avance les tensions qui se produisent dans une Construction", *Comptes Rendus* November, 1912.

obtaining Newton's interference fringes, to serve the same purpose, was also first suggested by Mesnager, and, later, was adopted by V. Tesart[169] and then by Frocht, to be abandoned finally on account of the mechanical difficulties of securing accurate results encountered in its application. The membrane method of making the same measurements was first suggested by L. Prandtl[170], and later was adopted first by Messrs. A. A. Griffith and G. I. Taylor[171] and then by Den Hartog.

Glass has been suggested as an ideal photo-elastic material because of its very low stress-optical sensitivity, and, when the determination of the direction of the principal stresses from the isoclinic bands, is the object in view. Many photo-elasticians still follow this practice and use a model made of a material of low sensitivity for the direction bands (isoclinics), and another, similar model, made of a material of higher sensitivity for the stress fringes (monochromatics). Unfortunately, the use of glass for models has been found to be impractical, primarily because of the difficulty of machining it into intricate shapes and also because it is extremely fragile under stresses of moderate intensity. The writer has recently found that a commercial synthetic plastic, having a stress-optical sensitivity only 1.25 times greater than that of glass, with a transparency rivaling that of glass, and possessing none of the disadvantages of the latter, can very advantageously replace the use of glass in models.

Considering the problem of determining stresses in models of machines and structures of the three-dimensional type, it has been suggested the previously silvered surface of a strip of photo-elastic material be cemented against the metallic surface of the prototype considered. By using polarized light reflected through and back from this cemented strip, the surface stresses (which, in general, are the largest of all across a given cross-section) can be determined from the isoclinics and monochromatics, in the usual manner. This idea was first suggested by Mesnager.[172] In 1936 the writer cited[167] the adaptability of liquid marblette as a coating on polished metallic surfaces. Successful results along this line have been thus far obtained only with very thin coatings, not of sufficient thickness to produce a fringe pattern under stresses of low intensity. Although such stresses can be measured by one of the compensation methods known to photo-elasticians (such as color matching, quartz-wedge compensators, Coker's tension bar, etc), these methods do not possess the ease and simplicity of the fringe-pattern method now universally in use. By additional coatings on the original one, the thickness of the skin layer can be increased to any desired amount, but the non-uniformity of the layer thus deposited may be objectionable. With the marblette liquid cement the writer has been able to produce a satisfactory bond between the surface of a metallic strip and a silvered surface of a piece of $\frac{1}{8}$-in. marblette. As this cement sets under room temperature conditions, initial strain effects in the cemented photo-elastic material are elim-

169 "Photo-Elasticite", *Revue d'Optique theorique et instrumentale*, Tome II, 1932.

170 *Physicalischer Zeitschrift*, Vol. 4, 1903.

171 Technical Rept., Advisory Committee on Aeronautics, Vol. 3, 1917–1918.

172 "Sur la détermination optique des tensions intérieures dans les solides à trois dimensions", *Comptes Rendus*, 1930, p. 190.

inated. Fig. 53 is a fringe pattern of stress in a model thus prepared, under a uniform bending moment applied to the metallic strip.

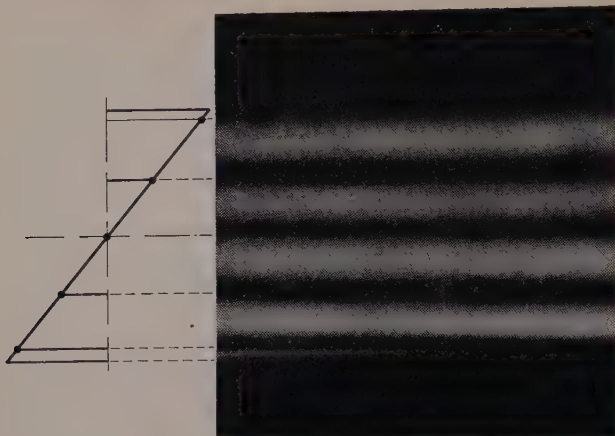

FIG. 52.—SURFACE STRESSES IN METALLIC STRUCTURES AND MACHINES FROM A STRIP OF PHOTO-ELASTIC MATERIAL CEMENTED TO THE PROTOTYPE.

Finally, it is interesting to note that the interferometer (or rather the interferometer-polariscope) as designed and built by the engineers of the U. S. Bureau of Reclamation is similar in principle as well as in construction to that first designed by H. Favre,[173] in Switzerland. A brief translation in English of Favre's paper concerning the theory and use of this instrument was made by the writer[174] in 1932.

H. D. HUSSEY,[175] M. AM. SOC. C. E. (by letter).[175a]—Structural engineering, as it involves the modern alloy steels, has developed to a highly technical state. The papers in this Symposium emphasize this fact. The use of these steels requires a better understanding of the phenomena of fatigue, stress concentrations at holes and fillets, buckling of plates, etc., as well as a better knowledge of the materials themselves. The writer wishes to emphasize the statements by the various authors concerning some of these factors.

Mr. Karpov gives a general outline of certain stress phenomena that are receiving much study, including an exposition of the theory of fatigue. He states that stability problems are less understood than problems of stress and that the tendency is to use higher factors of safety with respect to stability as compared to stress. It should be emphasized, however, that many stability problems encountered in structural design (such as the buckling of web-plates in a girder, plates under compression in a column, and outstanding flanges) have been solved,[176] and that the results have been embodied in the

[173] "Sur une Nouvelle Methode optique", *Revue d'optique theorique et instrumentale*, No. 8, 1929.

[174] "New Developments in Photo-Elasticity,,' *Journal*, Optical Soc. of America, May, 1932.

[175] Designing Engr., Am. Bridge Co., New York, N. Y.

[175a] Received by the Secretary February 17, 1937.

[176] "Elastic Stability of Plates," by Otis E. Hovey, M. Am. Soc. C. E., *Proceedings*, 36th Annual Convention, Am. Ry. Eng. Assoc., Vol. 36, 1935, p. 715.

specifications of the American Railway Engineering Association for Steel Railway Bridges (1935).

A problem of much importance is the solution of stresses in rigid-frame knees-braces. The over-sized model test reported by Mr. Templin is interesting as corroborating tests on a structural steel knee-section of similar shape made at the National Bureau of Standards, Washington, D. C., in collaboration with the American Institute of Steel Construction. The stress diagram on Line A-A', Fig. 14, across the corner, indicates a high concentration of stress on the inside, or curved, face and the absence of stress at the outside corner.

The problem of stress concentration about a hole in a plate has received much study, both theoretical and experimental.[177] It must be observed that in such studies the investigator assumes that the stresses are within the elastic range at all times. The phenomenon of stress concentration about a hole in a plate, when the theoretical stress exceeds the yield point, has not been explored so widely. Theory indicates that the stress at the edge of a hole in a wide plate may be three times the average stress on the net section. If the average stress is one-half the yield point the stress condition adjacent to the hole is shown in Fig. 54. (By "theoretical stress" is meant the stress

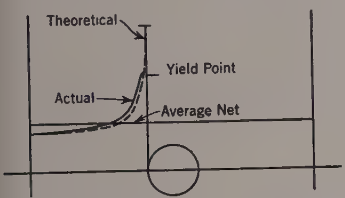

FIG. 54.—STRESS CONDITIONS ADJACENT TO A HOLE IN A PLATE.

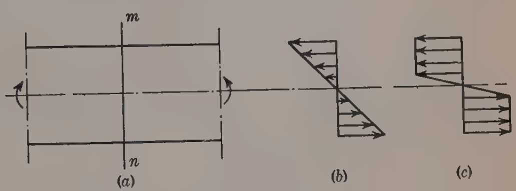

FIG. 55.

that would result if proportionality of stress and strain continued to any limit.) Since the maximum stress cannot be much greater than the yield point there is some re-adjustment of stress due to yielding of the material adjacent to the hole. Due to this yielding there results a small permanent set on release of the load. Repetition of this stress condition produces no further yielding, but a sufficiently large number of repetitions will lead to failure by fatigue.

The problem of riveted joints and stress concentration at rivet holes has received the consideration of several of the authors. A better understanding of this problem depends, among other factors, upon a more thorough knowledge of the theory of plasticity.

A problem of much importance is the question of proper, or permissible, ratio of yield point to tensile strength. Mr. Moisseiff calls attention to the fact that several of the new alloy steels have a high ratio. In the field of bridges and buildings, it has been considered that this ratio should not

177 "Theory of Elasticity", by S. Timoshenko, 1935, p. 75, and "Stress Concentration Produced by Holes and Notches", by A. M. Wahl and R. Beeuwkes, Jr., *Transactions*, Am. Soc. Mech. Engrs., Vol. 56, 1934, Paper APM-56-11.

exceed 70 per cent. It seems appropriate to inquire whether this, or any, limitation should be applied to ductile materials such as those under discussion.

The reason for a low ratio of yield point to tensile strength is to provide a reserve of strength against high concentrations of stress at joints, holes, fillets, etc. It is apparent at once that this reserve strength cannot provide against an increase of direct stress, either tension or compression, beyond the yield point, because, being the point of transition from an elastic to a plastic state, this is the useful limit of any member under those stresses. This reserve strength, therefore, can operate only under conditions of bending stresses beyond the yield point.

A study of stresses beyond the yield point shows that the bending resistance of a beam continues to increase after the stress in the outer fiber has reached the yield point, depending upon the make-up of the beam. For a rectangular section this increase may be as high as 50 per cent.[178] Fig. 55(*b*) shows the stress diagram on Section *m-n* of the beam in Fig. 55(*a*) when stressed to the yield point. The stress diagram of a beam stressed beyond the yield point can be represented with sufficient accuracy by Fig. 55(*c*). The deformation increases with the increase in bending moment on the beam, but the stress does not increase much beyond the yield point. When the total deformation is in the order of about ten times the yield-point deformation the resisting moment of a rectangular beam is nearly 50% greater than when stressed as in Fig. 55(*b*). This is a great reserve strength and is inherent in the beam by virtue of its being stressed by bending. It does not depend on the increase of tensile strength beyond the yield point, but only on the fact that the steel has ductility.

For example, examine the stress-strain diagram (Fig. 2) discussed by Mr. Karpov. When strains beyond the yield point exist, the amount of the strain is influenced very little by the shape of the stress-strain curve. It is apparent that the relief of stress concentrations is caused by the yielding of the material at the over-stressed parts and not by the numerical value of the tensile strength. Indeed, were it not for plastic elongation of the material there would be no relief. In such cases the important characteristic of the stress-strain curve is the amount of elongation rather than the increase in stress beyond the yield point. This leads to the conclusion that the physical properties of greatest importance are yield point and ductility (as measured by elongation and reduction of area), and that the tensile strength is secondary to these. Thus, if it is possible to produce a satisfactory ductility, the ratio of yield point to tensile strength is of minor importance.

The writer welcomes the papers by metallurgists in Section II of the Symposium. The paper by Messrs. Bain and Llewellyn is of interest to the structural engineer because of the explanation of the metallurgy of alloying elements. Their discussion of "Interpretation of Specimen Tensile Tests" is worthy of note. Such discussion, coming from those with a different point of view, should be of value to the engineer in appraising, correctly, the materials he uses.

[178] "Strength of Materials", by S. Timoshenko, Pt. I 1930, p. 237.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ECONOMIC DIAMETER OF STEEL PENSTOCKS

Discussion

BY MESSRS. R. A. MONROE, WILLIAM E. RUDOLPH, AND PETER BIER

R. A. MONROE,[19] M. AM. SOC. C. E. (by letter).[19a]—As noted by the authors, several formulas for computing the economic diameter of steel penstocks have previously been presented. All these formulas are based on the same fundamental principle, namely, that the most economic size of penstock is that for which its annual cost, plus the value of the power lost in friction, is a minimum. For any given set of conditions all will give practically identical results.

The authors have used the Scobey formula for flow in steel pipe lines for the stated reason that: "It is the most authoritative and the newest available." However, the use of the Weisbach formula,

$$Q = \frac{f\,L\,V^2}{2\,g\,D} \qquad (41)$$

(in which f is a friction factor) leads to a considerably simpler mathematical expression for the economic diameter with no appreciable sacrifice in accuracy, providing the factor, f, is properly chosen for the range of penstock diameters being considered. This simplification is particularly desirable when attempting to develop an expression which will include the hydraulic losses, h_h, other than those of friction, all of which are assumed to vary as the square of the velocity head.

The introduction of the load-factor loss curves to enable a quick approximation of the flow for which the economic diameter should be determined is a novel idea. It should be noted, however, that a hydro-electric plant which is tied into a large interconnecting system is seldom operated to produce an output that is proportional to the system load. When ample water is available the hydro-electric plant is usually operated at a high load factor at or near its point of best efficiency for most of the day, dropping off for only

NOTE.—The paper by the late Charles E. Votsch, M. Am. Soc. C. E., and M. H. Fresen, Assoc. M. Am. Soc. C. E., was published in November, 1936, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

[19] With TVA, Knoxville, Tenn.

[19a] Received by the Secretary January 23, 1937.

a short period when the system load is at a minimum; the peak load of the system is carried by steam plants operating on a very low load factor. During low-flow periods, this method of operation is reversed, particularly where facilities exist for forebay pondage, the hydro-electric plants being operated on the system peak whereas the base load is carried by steam.

In a hydro-electric plant having several units a reduction in output is usually effected by shutting down some of the units and continuing operation of the units remaining in service at the point of best efficiency. In this case the penstock loss is not substantially reduced providing the units are served by separate penstocks. A further point to consider is that peak power is always worth considerably more per kilowatt-hour than the average output of the system, as is indicated by the demand charge which is usually embodied in electric-rate schedules. Consideration of these facts will lead to the selection of a somewhat larger flow on which to base an economic design than would be chosen if the criterion selected was that flow for which the total friction loss is equal to the friction of the actual variable flow. The engineer can usually determine the proper basis of design for any particular case, but as a general proposition it appears that the economic design of a penstock should be based on a flow approaching that which will be utilized by the turbines at the point of best efficiency.

The value of power which should be used in the formula for economic penstock design cannot be determined readily but its exact value is not important since results are affected only in proportion to the seventh root of this factor. The same consideration applies to the ratio of fixed charges to total cost.

An easily applied formula for determining the economic diameter of a steel penstock is very desirable and convenient, particularly for preliminary studies. For the final design of a pipe line in which the various factors become definitely established, it seems quite desirable to make a detailed calculation of the cost and of the hydraulic losses for at least three sizes of penstock. Such a computation will permit a curve to be drawn showing the variation in cost of output in terms of diameter of penstock, which will afford a definite basis for the selection of the proper size. It also makes it possible to take into account many factors which do not lend themselves to formalization. It may perhaps be noted that since the cost curve for varying diameters is usually comparatively flat, the very real advantage of keeping initial costs to a minimum will often cause the selection of a pipe diameter considerably smaller than that shown theoretically as the most economical on the basis of estimates of the future value of power.

WILLIAM E. RUDOLPH,[20] M. AM. SOC. C. E. (by letter).[20a]—The application of one of the equations in this paper can be demonstrated with reference to a problem that often arises in penstock design—that of the single pipe line as compared to multiple pipe lines. Especially is this important in hydro-electric development in South America, where it is quite common, first, to install

[20] Chf. Engr., Mauricio Hochschild & Co., Ltd., Potosi, Bolivia.

[20a] Received by the Secretary January 26, 1937.

machines for utilizing only a portion of the potential power at a site, and, later, to install other machines to reach the full capacity of the normal water flow. Where there is to be a considerable lapse of years between the first and the latter construction stages of a hydro-electric project, the penstock is built of sufficient capacity for the initially installed units only. However, there are instances in which the second stage of development follows within a few years of the first, as, for example, in the development of a mine where the tonnage of ore to be treated increases rapidly. In such cases it is sometimes advantageous to build the penstock for the final project during the first stage of the development, because of the saving in cost of a single pipe line as compared to that of multiple pipe lines.

The writer has been trying to apply the formulas of this paper to a particular problem under consideration. He began by determining the economic diameter of penstock for the amount of power to be installed at once, and then for a single pipe line for the ultimate capacity (which in this case was exactly double the initial installation). The difference in ultimate capital investment between a single large pipe line and (in this case) two smaller pipe lines, was thus estimated. Such definite saving n yr hence, was converted back to its present value, ϕ, using the rate of interest applicable. Then, for the pipe line of the capacity of the present installation only, the annual cost of the penstock is defined by Equation (21). For the pipe line of a capacity of present, plus future, installation, the annual cost of the penstock (accurately enough for practical purposes), is:

$$E_t = E_f + E_h + E_p - \frac{\phi}{n} \qquad (42)$$

The smaller annual cost determines the more advantageous installation. In the case under consideration, n being only 2.5 yr, the greater capital expenditure for the full-sized penstock, with the initial installation, appeared to be justified.

PETER BIER,[21] ESQ. (by letter).[21a]—For the conditions cited, and within certain limitations, the economic diameter of steel penstocks and pump discharge lines can be determined by means of this interesting paper. Equations (11) and (26) have been derived by using Equation (6) for the calculation of the plate thickness corresponding to a given head. For low-head developments where the plate thickness is not a function of the head, but an arbitrary minimum established to satisfy the requirements of rigidity, fabrication standards, or long life, these equations will not give the proper answer.

An example of a welded-steel discharge pipe for a pumping plant (erroneously called "penstock" in the paper) will illustrate this point. Let $Q = 150$; $K s = 0.36$; $e = 0.71$; $F = 0.425$; $f = 0.13$; $b = \$0.0026$; $s_g = 13\,500$ lb per sq in.; $e j = 0.90$; $a = \$0.09$; $H = 150$ ft; $r = 0.07$; $i = 0.05$; $L = 1\,920$ ft; and, $t = 0.25$ in. (minimum).

[21] Engr., U. S. Bureau of Reclamation, Denver, Colo. Approved for publication by the U. S. Bureau of Reclamation.

[21a] Received by the Secretary February 18, 1937.

From Equation (11) modified for pump lines:

$$D = 0.50218 \sqrt[6.9]{\frac{0.36 \times 2\,044\,800 \times 0.13 \times 0.0026 \times 13\,500 \times 0.9}{0.09 \times 150 \times 0.07 \times 1.05 \times 0.71}} = 4.59 \text{ ft}$$

With the same data a detailed analysis, based on the total annual cost per foot of pipe, including annual fixed, operating, and maintenance charges, gives the following values, for the various diameters considered:

Diameter of pipe, in feet	Total annual cost, in dollars per foot of pipe
4.75	1.65
5.25	1.50
5.75	1.47
6.25	1.49
6.75	1.56

According to the foregoing data, the economic diameter would be 5.75 ft as compared to 4.59 ft obtained from Equation (11). The pipe was provided with a 0.25-in. plate thickness throughout, this being the minimum thickness recommended for that diameter in the chart used for the design of steel pipes by the U. S. Bureau of Reclamation.

In the foregoing example, Equation (6) would yield different plate thicknesses for the varying heads along the profile of the pipe line, with a maximum value of $\frac{3}{16}$ in., which, however, falls short of the practical requirements. Since the values included in Equation (6) enter into the final equations, the latter cannot be used for pipes under low heads, where values of t will fall below the minimum plate thicknesses established for reasons other than the requirements of head or the allowable tension in the plate.

On the other hand, if the plate thickness is given and the economical diameter determined from Equations (30) and (32), the same discrepancy arises as before. If t and D are substituted in Equation (6) to determine H, there will be two unknowns, namely, H and s_g, the latter being unknown for the reason that it was not used for the determination of t, which was assumed arbitrarily. The actual unit stress in the plate would have to be substituted for s_g to obtain the head, H, for which D is the economical diameter.

For the preceding detailed study the actual unit stress in the shell plate for a head of 150 ft, is,

$$s = \frac{5.75 \times 12 \times 150 \times 0.4335}{2 \times 0.25 \times 0.9} = 9\,970 \text{ lb per sq in.}$$

If $D = 5.75$ ft were obtained from Equations (30) and (32) for $t = 0.25$ in., the substitution in Equation (6) would give, for $s_g = 13\,500$ (the actual unit stress in the plate being unknown):

$$H = \frac{0.25 \times 2 \times 13\,500 \times 0.9}{0.4335 \times 5.75 \times 12} = 203.4 \text{ ft}$$

In other words, the authors' method would determine the economical diameter for a head of 203 ft instead of 150 ft.

For pipe lines supported on piers or columns with long spans, another error may be introduced in these equations by using the value, i, which consists of additions to the weight of pipe due to laps, butt straps, stiffener supports, etc. Values of i may vary from 0.05 to 0.25, depending on the span or type of support used. The trend in modern steel-pipe design is to use long spans between supports. This necessitates not only heavy stiffener rings and support brackets, but often a pipe shell of increasing thickness toward the supports so as to reduce the stresses in the shell portions adjacent to the supporting rings.

For the two cases cited herein a detailed study based on total annual cost curves for various diameters will be more satisfactory. It is problematical, of course, whether much economy can be derived from using the proposed method at all in preference to the detail analysis. The time required to become familiar with the lengthy equations and their derivations, the limitation of the method to cases in which the shell thickness is purely a function of the head, and where the span and support problems do not introduce undue weight additions, and, further, the chance of error in applying the equations, all mitigate against its economical use. It is possible, however, that if one is called upon to make numerous economic studies and becomes familiar with the method proposed by the authors, it may be used to advantage within the limitations outlined herein.

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DISCUSSIONS

RECLAMATION AS AN AID TO INDUSTRIAL AND AGRICULTURAL BALANCE

Discussion

By Joseph Jacobs, M. E. McIver, and C. S. Jarvis

Joseph Jacobs,[15] M. Am. Soc. C. E. (by letter).[15a]—One reads with admiration and, in the main, with approval, the forceful presentation made by the authors, of a phase of economics and sociology not often approached from the engineering angle. It is, in other words, the application of engineering analysis to certain economic and sociological problems which all too frequently in the past have measurably lacked that desirable form of analysis. It deals specifically with a phase of national development that is likely to become, if it is not already, an active political issue. The West understands and appreciates the benefits of reclamation, believes it to be a national asset of great importance, and if its value can be materially enhanced in the manner indicated by the authors, it is a matter of national economy and of national well-being in which every one has a vital stake—a consummation to be desired and to be striven for.

Some, no doubt, will feel that the authors are perhaps too optimistic as to the extent that major industries may advantageously be decentralized and transplanted to newly developed, or long existent, reclamation areas, and that the doctrine of self-sufficiency may be over-emphasized. That there are, however, distinct possibilities in that direction, and that there are some present tendencies toward industrial decentralization, there can be no question. Without intent to take issue with any of the authors' contentions, the writer desires to submit a few brief comments on their main thesis, as follows:

(*a*) A well balanced distribution of industry and agriculture is desirable and, unquestionably, diversification of occupation and decentralization of industry afford social advantages as pointed out by the authors. However, with the nation's splendid and steadily improving transportation and communication systems, complete balance for individual areas is not absolutely

Note.—The paper by Ernest P. Goodrich and Calvin V. Davis, Members, Am. Soc. C. E., was published in November, 1936, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[15] Cons. Civ. Engr., Seattle, Wash.

[15a] Received by the Secretary January 20, 1937.

necessary and, in many cases no doubt, not entirely desirable or possible from economic considerations. In general, both industrial and agricultural production should be fostered and expanded only in those localities which are best adapted to the particular industry and the particular agriculture in question. Development should not be unduly forced on a fetich of self-containment.

(*b*) If California is not entirely self-sustaining as to some industries; that is, if it is not a completely balanced economic unit, it is more than self-sustaining as to other industries for which it has a nation-wide and, to some extent, a world-wide market. Certainly, it would not desire to surrender these markets. Other States and other localities are similarly situated and that is, by no means necessarily, an unwholesome situation.

(*c*) More can be said for the need of national self-sufficiency than for State or local self-sufficiency. However, with a broadening of understandings and of amicable agreements between nations, and again with the steadily improving transportation and communication systems in the United States, even complete national self-sufficiency, in the aforementioned sense, is far from being a present-day imperative necessity. An active interchange of world trade is highly important to national prosperity and, similarly, an active interstate commerce is of high economic value. These desirable ends would be largely sacrificed if the doctrine of self-sufficiency is carried too far in practice.

(*d*) Important developments in the direction indicated by the authors are no doubt possible and desirable, but there is always economic danger in broad generalizations, and caution is essential in the application of new economic theories and doctrines. The fact is that each particular industry, and each particular locality, demands a detailed economic analysis by experts in the particular lines involved before a safe program in respect thereto can be outlined. That the authors recognize the essential correctness of this statement is evidenced in their discussion of "Location Factors."

It is noted (see heading "Cost of Water Supply"), that a uniform price of $4.05 per acre-ft is designated for all classes of water use. If the water charge for industrial and domestic use were increased three or fourfold (a charge that could readily be borne for those uses), it would permit a material decrease in the charge for irrigation water. Industrial gardens can stand a heavier water charge than general agriculture but, for either, a charge of $4.05 per acre-ft, which amounts to from $10.00 to $12.00 per acre per season, is rather heavy—much heavier than is customary on most irrigation projects. A variation of rates for the different uses would seem to be desirable.

M. E. McIver[16] (by letter).[16a]—It is re-assuring to find engineers supporting remedial measures, such as those suggested in this paper. The writer believes that the future may be considered safe when engineers, rather than sociological and political theorists, prescribe for the ills of society.

[16] Secretary, Am. Assoc. of Engrs., Chicago, Ill.

[16a] Received by the Secretary January 14, 1937.

The paper contains the kind of "medicine" for social and economic ills that the late Henry Arnstein, noted Consulting Chemist of Philadelphia, Pa., urged in the pages of every publication available to him. "Get the most out of natural resources", he pleaded, and his doctrine was more widely accepted in the thirty foreign countries to which he was special adviser in conservation policies, than in his own wealthy and profligate country.

The authors should add to their treatise an appeal for the development in the areas described, not only of branches of old established industries, but of new manufacturing agencies to produce—from surpluses, and from waste material—synthetics suitable as substitutes for material now imported. That was the final treatment in Dr. Arnstein's formula for changing a wasteful nation into an independent and stable power.

Much of the appalling extravagance of the last few years will be compensated and the future tax burden not only justified but "written off", if the policies outlined in this paper can be put into practice; and great social values can thus be developed from mere (but inconceivably expensive) power projects. Engineers should teach the politicians to go on to the ultimate Arnstein project and demand that, in addition to reforestation, for example, they learn all the synthetic possibilities of every splinter of every tree that is cut; that instead of merely making more sugar to compete with "reciprocity" sugar, the new industries be encouraged to develop fuel alcohol; and, that in some of these areas, the dyestuffs and fertilizers and hundreds of other products now imported, may be produced by engineering research magic from surplus and waste. Engineers will have occasion for greater pride in their leadership in the field of conservation than in the construction of the tallest buildings or the biggest bridges, which seem to be the largest scale production now attributable to engineering knowledge and skill. The latter utilities are evidence of genius, but conservation progress is genius plus patriotism.

C. S. JARVIS,[17] M. AM. SOC. C. E. (by letter).[17a]—The clearly expressed views presented in this paper are both impressive and convincing because of the sound logic used, the practical examples cited, and the reasonable conclusions reached. Such examples and principles must be relied upon largely for future guidance in the decentralization of industry, orderly diversification of labor, and co-ordination of agriculture with other essential and basic industries.

Pioneers and frontiersmen generally have had to develop ingenuity in shifting for themselves, to become both self-reliant and reasonably self-sustaining, and thus to become largely immune to the hardships ordinarily attending isolation. Over-specialization of industry in crowded centers has had the opposite trend and effect; any reasonable means for bringing about decentralization of industry and diversification of labor, especially among rural surroundings and in more or less closely related branches of agriculture, should be encouraged.

[17] Hydr. Engr., SCS, Washington, D. C.

[17a] Received by the Secretary February 12, 1937.

Too often such motives have been announced, the colonies and industries established, the land subdivided and disposed of at rising prices, only to suffer collapse when it has been demonstrated that the proposed industries cannot thrive under local conditions as to raw materials, power, labor, water supply, or market. In other instances, smelters and mills for the reduction of ore have been established in rural communities, only to be shut down and removed after conclusive demonstration that the by-products of flues and the odors or dust attending the processes, and the effluents from drains are objectionable and harmful to various forms of life. Many communities have been brought to the realization that they have paid too high a price for industrial activity and temporary prosperity, when they discovered the deleterious effects of pollution in atmosphere, streams, pastures, fields, orchards, and in the soil itself. Other objectionable effects have attended the establishment of industries with characteristically unpleasant and noxious odors.

If the well-matured studies and procedure associated with community planning are utilized in connection with both reclamation and the various industries that are considered in an effort to promote a stabilized industrial activity, there is reason to expect attainment of the desired objectives, with avoidance of the costly errors so often experienced heretofore. Long-range planning implies the consideration of all lines of activity and growth that may be reasonably foreseen or imagined, with adequate provision for each essential element of the industrial system along with agriculture and its ramifications, and finally, adjustment and reconciliation of their divergent requirements.

The citizens of the country must recognize and adjust themselves to the inevitable trend away from the smallest industrial units—the "single run-of-stone" grist-mill, now generally supplanted by roller mills; the water-driven, seasonally operated saw-mills and planing mills or wood-turning shops, ordinarily giving way to electrically operated all-year plants; the small textile factories unable to compete with the larger units or associations, and, therefore, readily absorbed during dull periods at bargain prices; the small tanneries, cement works, lime-kilns, foundries, glass works, canneries, dehydration plants for fruit and vegetables; the small creamery, ice-plant, storage warehouse, laundry, and indeed nearly all the ordinary commercial and manufacturing enterprises. The increasing complexities of accounting, inspection, chemical analysis and testing, research laboratories, water supply, fire protection, storage, buying of raw materials at favorable prices, marketing of the finished product, smoke abatement, industrial and household waste disposal, and maintaining good-will of customers and of public officials—all these have seemingly grown beyond the capacity of an individual or of a small group. They demand a technical staff with background and experience as broad as the problems they will be likely to encounter. The dependence of a highly trained staff, the operators, and their families, upon the stability of the enterprise calls for the maintenance of a substantial reserve of both raw materials and finished products, as well as substantial credit and liquid assets, to bridge over periods of depression.

For those occasional instances where a small industry can thrive, such as for corn-meal or other feed-grinding units at a centrally located farm, convenient to dependable and low-cost power, there should be an attractive future as long as the quality of the product is conscientiously maintained according to acceptable standards. Service to certain farms with heavy tonnages may be rendered to best advantage by means of portable grinding units. As such work is usually seasonal in character, and follows the main harvests of grain, it could well be one of the auxiliary activities maintained in connection with a well-equipped farm.

Other services, trades, and specialized work are normally in demand wherever a community establishes itself and has a reasonable assurance of continued growth and prosperity; but none of them has so nearly a guaranteed supply of raw materials nor so wide a market demand as have those branches of agriculture which furnish foodstuffs within ready access of population centers. The important part of the unemployed families' living supplied by the industrial co-operative garden projects, as described so interestingly by the authors, vividly illustrates the nearly independent status of rural home owners with small acreages and skill, industrious habits, and enthusiasm for agricultural enterprises appropriate to the region. To such a citizen, trained to thrifty ways and inclined to make some provision for the future, occasional interruptions of industrial employment mean merely a better opportunity to concentrate on his gardening or such other activities as he has established, more rigorous reduction of expenditures, depletion of savings, or a combination of such measures. He is not facing unemployment, but rather the problem of adaptation to available employment.

It must be recognized that many individuals and families are quite unfitted by natural inclination, training, and habits of life to become contented and dependable members of rural communities relying mainly upon agricultural pursuits. Early Colonial history and subsequent frontier life in this country afford many examples of failure because of unwillingness or inability to forego the use of seed-grain and vegetables during periods of privation, when such temporary appeasement of hunger plainly meant another season of famine or abandonment of the venture, admitting defeat at their own hands and under the stress of circumstances.

Granting that improved means of communication and the recognition of society's responsibility to aid individual members during emergencies have radically changed the outlook for struggling colonists and "back-to-the-soil" devotees, some vestige of that self-restraint, control over appetites, and long-range planning for economic stability must be maintained if resettlement projects are to prosper in an approach to their main objectives. Among these are decentralization of industry, diversification of labor, co-ordination of agriculture with other basic industries, thereby establishing a positive defense against the recurrent depressions, temporary collapse of property and security values, disorganization, distress, discouragement, and attendant ills.

On many reclamation projects where irrigation is the main dependence for crop production, snow surveys afford a satisfactory index to the season's

water supply. If reservoirs provide considerable carry-over from years of surplus, the inclination is to draw a minor portion of such reserve during any one season. During years of extreme drought, and before adequate reservoirs are provided, it occasionally happens that the limited irrigation supply is required mainly within orchard and small garden tracts. The main recourse for the growing of forage and grain crops, under such circumstances, has been dry-farming, resulting in the improvement of methods and the pronounced extension of areas within which it has proved moderately successful, year after year. Even such persistent weeds as the Russian thistle provide excellent, succulent pasturage as long as they are kept closely grazed; and the mature plants have much nutriment available for livestock if included with ensilage, or if moistened thoroughly to soften the thorny spikes.

Unfortunately, mounting tax rates have become more and more burdensome to the land-holder and home owner, and this item is not subject to curtailment during periods of depression and retrenchment. If the assessed valuation of a county is reduced, the requirements for revenue result in a proportionate increase of tax levies, thus leading to delinquencies and ultimate forfeiture unless normal market demands and prices for agricultural products are restored without undue delay.

Either depression or prosperity incident to agriculture seems to be reflected throughout the national economic structure sufficiently to make the co-ordination and stabilization of its various branches, along with other basic industries, one of the foremost national problems of to-day.

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DISCUSSIONS

STRUCTURAL ANALYSIS BASED UPON PRINCIPLES PERTAINING TO UNLOADED MODELS

Discussion

BY MESSRS. L. J. MENSCH, AND FREDERICK SHAPIRO

L. J. MENSCH,[5] M. AM. SOC. C. E. (by letter).[5a]—The many papers on rigid-frame analysis that have been published by the Society have evidently intrigued the author, an engineer of exceptional ability in the art of the design of hyperstatic structures and of extended experience, to contribute his share to the excitement, or "talk-fest", on the subject of rigid-frame analysis.

The honor of having first simplified the analysis of continuous structures belongs to Clapeyron[6]. For the development of his famous formulas, Clapeyron used a method which in the United States has lately been called the "slope-deflection theory." These "slope-deflection formulas", are in common use for all kinds of problems in the well known textbook of Dr. F. Grashof[7], and in many textbooks issued before and afterward.

Professor Grashof treats[8] the problem of a girder, $A\,B$, partly fixed at the supports by the moments, M_A and M_B, having the slopes, α and β, of the elastic line at A and B, and affected by an uniform load, W, and a concentrated load, P, distant a and b from the ends, A and B, and gives the following formulas:

For moment:

$$M_A = -\frac{P\,a\,b^2}{L^2} - \frac{W\,L}{12} + \frac{2\,E\,I}{L}\,(2\,\alpha - \beta) \quad \text{.............}(51a)$$

and,

$$M_B = -\frac{P\,a^2\,b}{L^2} - \frac{W\,L}{12} + \frac{2\,E\,I}{L}\,(-\,\alpha + 2\,\beta) \quad \text{..........}(51b)$$

NOTE.—The paper by Otto Gottschalk, Esq., was published in January, 1937, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[5] Civ. Engr. and Constructor, Chicago, Ill.

[5a] Received by the Secretary, February 5, 1937.

[6] *Comptes Rendus*, Vol. 45, Paris, 1857.

[7] "Theorie der Elasticität und Festigkeit," Second Edition, Berlin, 1878.

[8] *Loc. cit.*, p. 78.

and, for shear:

$$V_A = \frac{P\,(3\,a + b)}{L^3}\,b^2 + \frac{W}{2} + \frac{6\,E\,I}{L}\,(-\,\alpha + \beta) \quad \ldots\ldots\ldots (52a)$$

and,

$$V_B = \frac{P\,(3\,b + a)}{L^3}\,a^2 + \frac{W}{2} + \frac{6\,E\,I}{L}\,(\alpha - \beta) \quad \ldots\ldots\ldots (52b)$$

He shows[9] that α and β must be replaced by $\theta_A - R$ and $R - \theta_B$, when Point A is lifted a distance, $R\,L$, over Point B during the deformation of the girder. He also shows[10] that, when $M_B = 0$, Equation (51*b*) becomes:

$$E\,I\,\beta = \frac{P\,a^2\,b}{4\,L} + \frac{W\,L^2}{48} + \frac{E\,I\,\alpha}{2} \quad \ldots\ldots\ldots (53a)$$

and, from Equation (51*a*):

$$E\,I\,\alpha = M_A + \frac{P\,a\,b\,(b + 0.5\,a)}{3\,L^2} + \frac{W\,L^2}{24} \quad \ldots\ldots\ldots (53b)$$

When both M_A and M_B are zero, Equations (51) yield the well known formulas:

$$E\,I\,\alpha = \frac{P\,a\,b\,(a + 2\,b)}{6\,L} + \frac{W\,L^2}{24} \quad \ldots\ldots\ldots (54a)$$

and,

$$E\,I\,\beta = \frac{P\,a\,b\,(b + 2\,a)}{6\,L} + \frac{W\,L^2}{24} \quad \ldots\ldots\ldots (54b)$$

Girders of varying section are also treated in this textbook.

In addition to Grashof, there is at least one other old elementary textbook, that of Wilhelm Keck[11], in which may be found all the slope-deflection formulas in use for the solution of continuous structures. The first textbook which the writer can now remember to have treated rigid-frame, multiple-storied structures is that of Professor E. Winkler[12] in which the "slope-deflection" formulas were freely used.

After this revelation of the status of the "slope deflection theory", it is to be hoped that enthusiasts will not further clutter up American literature with the incorrect statement that the "slope deflection theory" is an American development, improvement, or invention, except in so far as the coinage of the phrase, "slope deflection," is concerned.

How did it come about that these important and simple earlier methods have been nearly forgotten by American and European engineers? It seems to the writer that teachers on any subject, in order to acquire at least a

[9] "Theorie der Elasticität und Festigkeit," Second Edition, Berlin, 1878, p. 101.

[10] *Loc. cit.* p. 83.

[11] "Vorträge über Elasticitäts-Lehre," Hannover, 1893.

[12] "Vorträge über Brückenbau," Wien, C. Gerold, 1875.

quasi-reputation, must produce something new, even if it is not better than the old art, and their students spread the new "light", and, presto, a new philosophy is in fashion. So it was with Mohr's principle of virtual velocities which was soon put out of fashion by Manebrea's and Castigliano's principle of least work (originally invented by Daniel Bernouilli in the Eighteenth Century) which principle found an enthusiastic and able propagandist in Professor Müller-Breslau and, through him, came to be used all over the world. Not so many years passed, however, before it was discovered that this intriguing method was practical only for comparatively simple problems, and could not be used by busy men for highly hyperstatic structures, and the pendulum (but not the habit, so easily) swung backward again and various types of "slope deflection" methods were re-invented.

One of these is the principle of fixed or conjugate points, so well treated by W. Ritter[13] and greatly elaborated for framed continuous structures, consisting of straight and arched members, by A. Strassner[14]. The author's paper is nothing but a variation from Strassner's treatment of this subject. Strassner uses the angles, whereas the author uses the intercepts of the tangents on the verticals at the opposite support, an unnecessary complication. The author has treated only the simplest cases in Strassner's work and has not made the slightest improvement in the formulas or in their derivation. Like Clapeyron, Ritter, and Strassner, he proposes to analyze a structure by beginning at one end and going over its entire length in order to determine the relative angle changes that are necessary for the computation of the unknown bending moments. This is clearly too large and entirely too impractical a task for the large structures that American engineers must design so often, and further simplifications are absolutely necessary so that engineers may be able to arrive at more precise results with comparatively little effort. Strassner gives a hint in one of his examples of an attempt at simplification, but does not go far enough as he has still to go through the entire structure in order to solve that part of it which may require more precise treatment.

How to solve any girder or column in a large structure quickly and with sufficient accuracy, the writer has shown elsewhere[15]. He obtained a sufficiently close estimate of the values of α and β in Equations (51) by setting,

$$\alpha = \frac{-M_A}{\Sigma\, m_A\, K_A} \qquad \text{(55a)}$$

and,

$$\beta = \frac{-M_B}{\Sigma\, m_B\, K_B} \qquad \text{(55b)}$$

in which m_A and m_B are certain factors (given in that discussion) and K_A equals the ratio of $\frac{I}{L}$ for each member adjoining A, except $A\,B$.

[13] "Graphische Statik," Zurich, 1900.
[14] "Neure Methoden", Berlin, 1916.
[15] See p. 578.

The solution of Equation (51) after the values of α and β from Equations (55) are introduced in them, gives the two simple formulas for a girder with an uniform load, W:

$$M_A = -\frac{W L}{4} \frac{1 + 2 N_B}{4 (1 + N_A) - 1} \quad \text{................(56a)}$$

and,

$$M_B = -\frac{W L}{4} \frac{1 + 2 N_A}{4 (1 + N_A) (1 + N_B) - 1)} \quad \text{............(56b)}$$

in which,

$$N_A = \frac{3 K}{\Sigma m_A K_A} \quad \text{.......................(57a)}$$

and,

$$N_B = \frac{3 K}{\Sigma m_B K_B} \quad \text{.......................(57b)}$$

The author's treatment of linear displacement is not as simple as that of Strassner and contains the same limitation in the sense that it is strictly applicable only to one-story structures, or to frames which remain vertical in the stories above and below the story which has been displaced, a case that never occurred in the writer's experience. The actual problem is to find the restraint at A and B when the entire structure is deformed as a girder due to the horizontal load acting at A, which is really a difficult problem; possibly, the author may be able to reveal some simplifying suggestions by experimentation. The problem of side-sway in a non-symmetrical single-story bent of two columns can be more easily solved by Strassner's method, which is really only a variation of Clapeyron's method.

The writer has heard engineers state that a method of analyzing highly hyperstatic structures which gives the unknown bending moments within 5% to 10% of their ideal values is necessary. No method, however laborious, known to-day can give such a close result. First, girders and columns have dimensions of from 6% to 20% and more of the spans and story heights, and not enough tests have been made to establish how near the designer guesses when he uses clear spans instead of theoretical spans. Although Strassner shows how to consider the shortening of columns by girders, he uses theoretical spans in his examples.

Another serious disturbance is the variation of the modulus of elasticity, E, under high or long-continued loadings. After all, the final intent of these analyses should be the establishment of the factor of safety of designs against failure or undue deformations. All formulas are derived by the assumption of a constant value of E.

Under intense loadings the modulus of elasticity of the highly stressed part of the member decreases rapidly, and if it is the end part of a restrained beam, for example, this diminution of E will have the effect of decreasing restraint and will modify the moments both in the center of the span and at the ends.

The center moment will be increased and the end moments decreased. No clear-cut tests have been made to estimate readily, the decrease of restraint. A very able treatment of this phase of the subject has been offered[16] by E. Mirabelli, M. Am. Soc. C. E.

That much has yet to be learned about the deviation of Hooke's law in such a standard material as structural steel is made evident by tests reported by the National Bureau of Standards[17]. A unit load of 31 000 lb per sq in. was imposed on two heavy stiff columns, 159 sq in. in cross-section and 24 ft long. The shortening was found to be 0.0016 in. per in.; and, therefore, the secant modulus of elasticity was only $\frac{31\,000}{0.0016} = 19\,000\,000$ lb per sq in. The moduli of elasticity decreased when stresses increased beyond 24 000 lb per sq in. It would have been of great importance to know what they would have been had the load remained for a month or a year.

That the deviation of E in reinforced concrete members plays an important rôle was shown by the tests to destruction of Mikishe Abe, M. Am. Soc. C. E.[18], on rigid-framed structures. The ultimate strengths were at least 10% larger than those computed by current analysis when clear spans were used. For this reason highly complicated formulas or processes which require long study will be of limited value in this field.

FREDERICK SHAPIRO,[19] JUN. AM. SOC. C. E. (by letter).[19a]—The method of analysis presented in this paper is an excellent variation of the moment-distribution principle. The underlying theory is the same as that of moment distribution, although the problem is viewed from a different angle. When one part of a frame is rotated and the effects of this motion are traced by multiplying the original rotation by factors the values of which depend upon the rotation resistance of the constituents of the frame, the method of moment distribution is being used, even if angular changes instead of bending moments are involved.

The formulas of the paper may readily be converted into more useful ones. Following the notation of the paper,

$$C = \frac{1 - 2m}{2 - m} \qquad \text{.............................(58)}$$

and instead of giving the ratio of the angular rotations of the ends of unloaded spans, Equation (58) gives the ratio of the end moments, in which C is the transmission coefficient introduced by R. C. Brumfield, M. Am. Soc. C. E., who developed an analysis of statically indeterminate structures based upon it.[20] It is a very useful value for computing end moments, even for

[16] *Proceedings*, Am. Soc. C. E., December, 1936, p. 1625.

[17] *Journal of Research*, National Bureau of Standards, *Research Paper 873*, March, 1936.

[18] *Bulletin 107*, Eng. Experiment Station, Univ. of Illinois, 1918.

[19] Rodman, Borough Pres. of Manhattan, New York, N. Y.

[19a] Received by the Secretary February 8, 1937.

[20] "The Analysis of Monolithic Structures by Transmission Coefficients", by Frederick Shapiro, *Concrete*, May, June, July, and August, 1936; see, also, "Moving Loads on Beams with Restrained Ends", by R. C. Brumfield, M. Am. Soc. C. E. (Complete manuscript filed for reference in Engineering Societies Library, 33 West 39th Street, New York, N. Y.)

loaded spans, and may be found from the formula:

$$C = \frac{Q}{2\,Q + F_r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (59)$$

in which $Q = \frac{1}{k} = \frac{L}{I}$, and F_r is a term called a restraint factor, and is equal to $\frac{1.5}{K_B}$. It is given by,

$$F_r = \frac{1}{\frac{1}{Q_1\,(2 - C_1)} + \frac{1}{Q_2\,(2 - C_2)} + \frac{1}{Q_3\,(2 - C_3)} + \ldots\ldots} \quad \ldots . (60)$$

A term of the form, $Q\ (2 - C)$, is called a stiffness index. Obviously, each end of a beam has its own transmission coefficient, restraint factor, and stiffness index $\left(= \frac{2}{3\ S_1} \text{ or } \frac{2}{3\ S'_1} \right)$. The stiffness indices found in the denominator of Equation (60) are those for the ends of all the beams meeting at a joint, except that of the beam end for which the restraint factor is to be found.

The transmission coefficient varies from zero for pin-ended beams to one-half for fixed-end beams of constant moment of inertia. In the interior of interlocking frames it may be approximated by the following method: Compute the ratio of the sum of the $\frac{I}{L}$ - values for the other members at the joint to twice the sum of the $\frac{I}{L}$ - values of all the members at the joint. If this value is between zero and 0.35, add 0.02; if it is between 0.36 and 0.43, add 0.01; and if it is greater than 0.43, add nothing to obtain the coefficient with less than 1% error. This value may be corrected later.

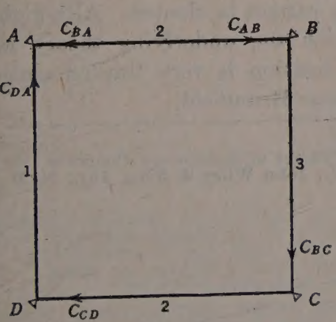

FIG. 19

For example, C_{AB}, in Fig. 19, will be computed. The large numbers on the spans are Q - values of $\frac{L}{I}$ - values. Approximately, C_{AB} $= \frac{K_{BC}}{2\,(K_{AB} + K_{BC})} + (?) = \frac{\frac{1}{3}}{\left(\frac{1}{2} + \frac{1}{3}\right)} + (?) = 0.20 + 0.02 = 0.22$. Using Equation (59) $C_{DA} = \frac{Q_{DA}}{2\,Q_{DA} + Q_{AB}\,(2 - C_{AB})} = \frac{1}{2 \times 1 + 2\,(2 - 0.22)} = 0.180$; $C_{CD} = \frac{2}{2 \times 2 + 1\,(2 - 0.180)} = 0.343$; $C_{BC} = \frac{3}{2 \times 3 + 2\,(2 - 0.343)} = 0.322$;

and, finally, $C_{AB} = \dfrac{2}{2 \times 2 + 3\,(2 - 0.322)} = 0.222$, which is correct to three decimal places. The change in C_{DA} would be indistinguishable on a slide-rule.

If a bending moment of 1 000 ft-lb were introduced at Point A, it would flow around the frame, becoming 1 000 C_{AB} at Point B, or 222 ft-lb; $222 \times 0.322 = 71$ ft-lb at Point C; $71 \times 0.343 = 24$ ft-lb at Point D; and, $24 \times 0.180 = 4$ ft-lb at Point A, and would continue around the frame again in the same manner.

In computing end moments for loaded spans, the transmission coefficients receive their greatest use. Formulas have been derived for end moments[20] and these involve only constants dependent upon the type of loading, the position of the load on the span, and the transmission coefficients for the span.

The writer and Professor Brumfield have developed charts to give transmission coefficients, restraint factors, and torque splits and moment splits at joints. They have arranged tables to reduce computations to a minimum and to simplify the work. Professor Brumfield has derived formulas for locating loads to produce maximum moment at any given point in a span or under any given load. He has also modified his formulas for use with clear spans and generalized them to include beams of variable moment of inertia, providing the neutral axis may be considered to be a straight line.

The writer has deduced mechanical methods for the determination of transmission coefficients, comparable to those for finding the carry-over factors in the method of distributing fixed-end moments by successive approximations. It might be noted here that the transmission coefficients, summing up by formula the converging series of the Cross method,[21] do in one step what the fixed-end method does by successive approximation. Many short-cuts have been found that may be applied without restricting the generality of Professor Brumfield's formulas. A method of checking computations for continuity, besides the obvious check at each joint for $\Sigma M = 0$, has also been developed[22] by the writer.

The usefulness of direct moment distribution cannot be denied. Although Dr. Gottschalk has presented a method in which the underlying theory is based on a simple conception, its practical application is very limited compared to the system of analysis evolved by Professor Brumfield.

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[21] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 1.

[22] Based on method given on p. 91 of "Continuous Frames of Reinforced Concrete", by Hardy Cross and N. D. Morgan, Members, Am. Soc. C. E., John Wiley & Sons, Inc., N. Y., 1932.